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Passaic River Tunnel Diversion Model Study

Report 1
Passaic River Flood Protection Project, Upper Basin Sedimentation Study

by David D. Abraham, William A. Thomas

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Prepared for U.S. Army Engineer District, New York

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Waterways Experiment Station
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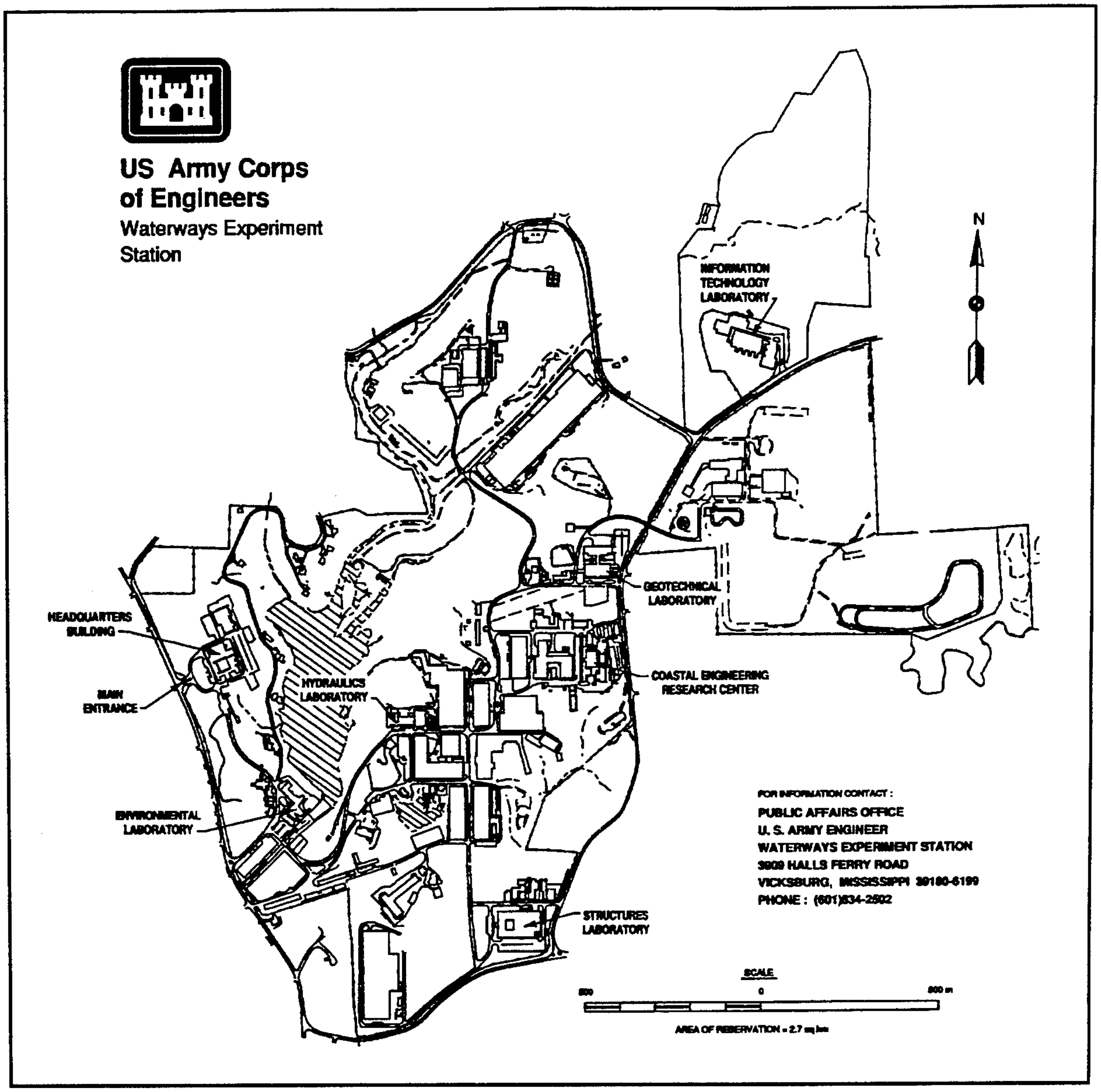

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Preface

This study was conducted as a part of the Passaic River Flood-Control Project by personnel of the Hydraulics Laboratory (HL) of the U.S. Army Engineer Waterways Experiment Station (WES) in cooperation with the U.S. Army Engineer District, New York.

Project manager for the New York District was Mr. John Bianco. Principal Investigator for WES was Mr. David D. Abraham, Rivers and Streams Branch, Waterways and Estuaries Division, HL. Mr. William A. Thomas of Mobile Boundary Hydraulics, Inc., Clinton, MS, was contracted as a sedimentation specialist for the study. Technical support was provided by Ms. Peggy H. Hoffman, Rivers and Streams Branch. The study was conducted between February 1994 and April 1995.

This report, Report 1, describes the modeling of the upper basin sedimentation characteristics and responses under base and plan conditions. Report 2 describes the sediment transport modeling for the estuary performed to evaluate changes in erosion and deposition patterns within the estuary in response to the modified hydrodynamics. Report 3 describes the modeling performed in support of the three-dimensional water quality modeling of the lower portion of the project area. Report 4 describes the field data collection and analysis efforts performed by WES in support of the sediment transport and hydrodynamic models. Report 5 describes the water quality modeling effort for the estuary.

This report was written by Messrs. Abraham and Thomas. The study was conducted under the general supervision of Messrs. Frank A. Herrmann, Jr., Director, HL, and Richard A. Sager, Assistant Director, HL, and under the direct supervision of Messrs. Michael J. Trawle, Chief, Rivers and Streams Branch, and William H. McAnally, Jr., Chief, Waterways and Estuaries Division.

At the time of the publication of this report, Dr. Robert W. Whalin was Director of WES. COL Bruce K. Howard, EN, was Commander.

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1 Introduction

Description of the Project

Passaic River and its watershed are located in northeastern New Jersey and southeastern New York. The basin is about 2,421.64 sq km (935 square miles), and drains the Passaic, Whippany, Rockaway, Pequannock, Wanaque, Ramapo, Pompton, and Saddle Rivers. Details of the watershed descriptions can be found in the report "Passaic River Flood Protection Project, Preconstruction Engineering and Design."[1] Figure 1 shows the project features.

The area with the greatest flooding potential is the Pompton River. To alleviate this problem, the construction of a 12.80-m- (42-ft-) diameter tunnel is proposed. This tunnel will have a main inlet at the confluence of Pompton, Ramapo, and Pequannock Rivers, and a spur inlet on the Passaic River just south of Two Bridges. The tunnel will empty into Newark Bay. The design will divert the peak of the floodwaters underground from the upper Pompton River and Passaic River at Two Bridges to Newark Bay.

The diversion through the tunnel will lower water-surface elevations in many of the project reaches. Although this is desirable from the standpoint of flood protection, it could alter sedimentation patterns. Such alterations are especially important near the tunnel inlets, in the vicinity of the Great Piece Meadows wetland, and along the Pompton River. This portion of the study addresses these concerns.

Purpose and Approach of Sedimentation Investigation

This study was carried out in support of the Passaic River Flood-Control Project. The purpose of the study was to determine the channel responses of the rivers affected by the planned flood diversion tunnel inlets. This

[1] U.S. Army Engineer Districts, Savannah and New York. (1992). "Passaic River Flood Protection Project, preconstruction engineering and design," Savannah, GA, and Hoboken, NJ.

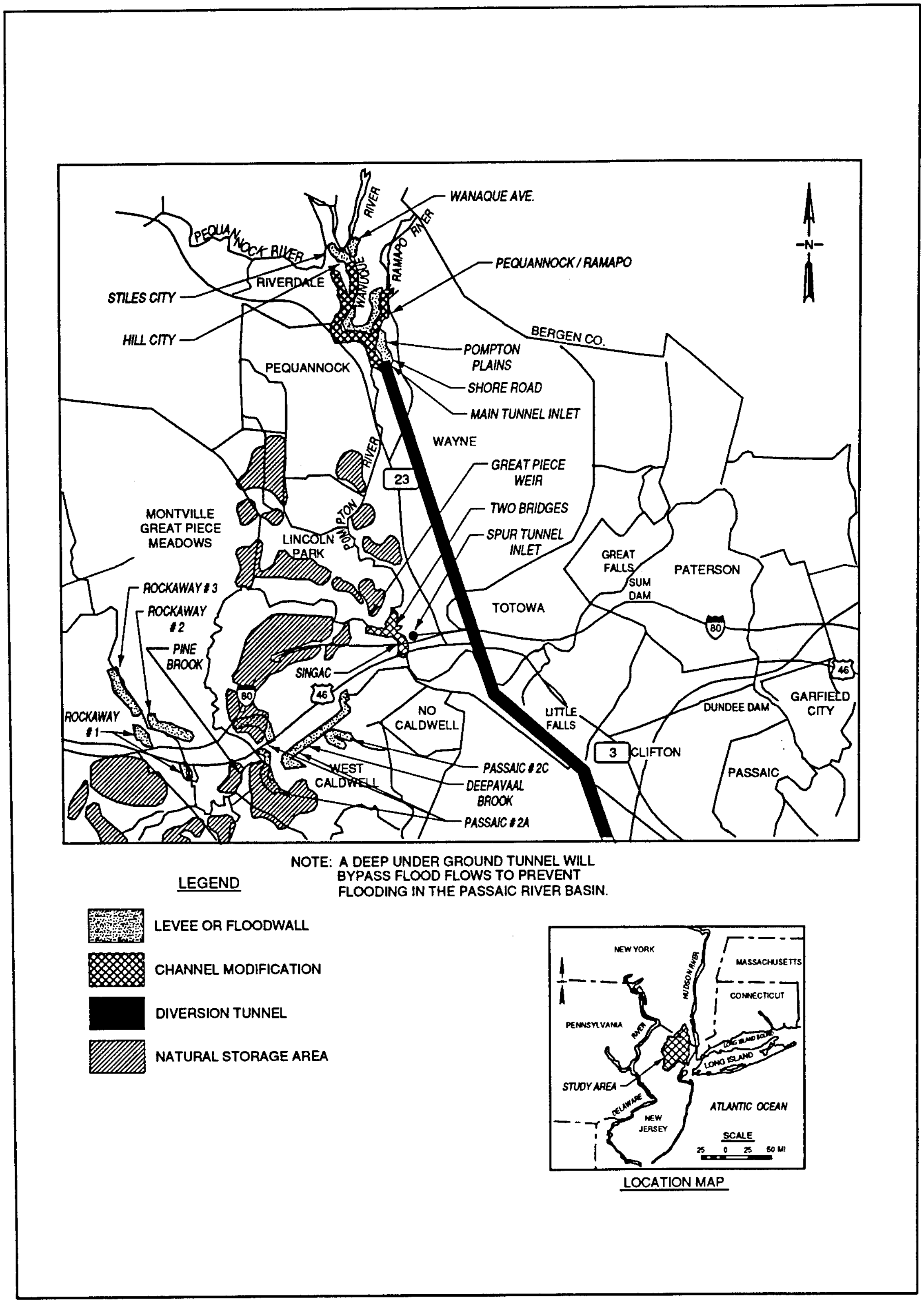

Figure 1. Passaic River Flood Protection Project, Pompton River/Passaic River Dual Inlet Tunnel

information will be used to evaluate the trap efficiency of the tunnel, to assist in locating structures in the rivers, and to identify areas that might require maintenance due to scour and deposition.

Prior to the study, it was known that the upper reaches of the Passaic River Basin contain branched river networks whose peak flows occur at different times and locations. These phased peaks can result in reverse flows in some reaches. Bed material also varies from reach to reach.

HEC-6, a one-dimensional multigrain sediment transport code, was selected for the major portions of the study. At the time the study was initiated, HEC-6 had no provision for closed loop or reverse flows. Revisions to the code had to be made during the course of the study so that reverse and branching flows could be modeled. In order to simulate the phasing of flow peaks, the flow hydrographs from UNET, a one-dimensional unsteady hydraulic model, were taken as hydraulic input to the HEC-6 model. The UNET runs were made by the U.S. Army Engineer District, Savannah, for the U.S. Army Engineer District, New York, and supplied to the U.S. Army Engineer Waterways Experiment Station (WES).

At the beginning of the project, several tasks that would need to be accomplished and the sequence that they should follow to conduct a complete and successful project study were identified. All available existing data, maps, and model runs were reviewed to become familiar with the project. A field trip to the site was conducted to get a first-hand view of bed forms, general channel characteristics, vegetation, and network orientation. At that time contact was made with the District personnel and U.S. Geological Survey (USGS) data collection crews to glean additional information that might be useful. All the information and data were then assessed to determine what was immediately useful and what would require additional processing for the ensuing HEC-6 model runs. A significant amount of data file manipulation was necessary to make use of geometric and hydrologic data. This was accomplished by writing new codes and modifying others. The HEC-6 model network was then developed as shown in Figure 2 and appropriate geometric, sediment, and hydrologic input files created. The model was then verified to hydraulic and sediment field data where possible, and otherwise to UNET hydraulic data for water-surface elevations. First base and then plan project conditions were run, with the final intent being to learn about any differences between these two runs. A more detailed description of the study and findings follows.

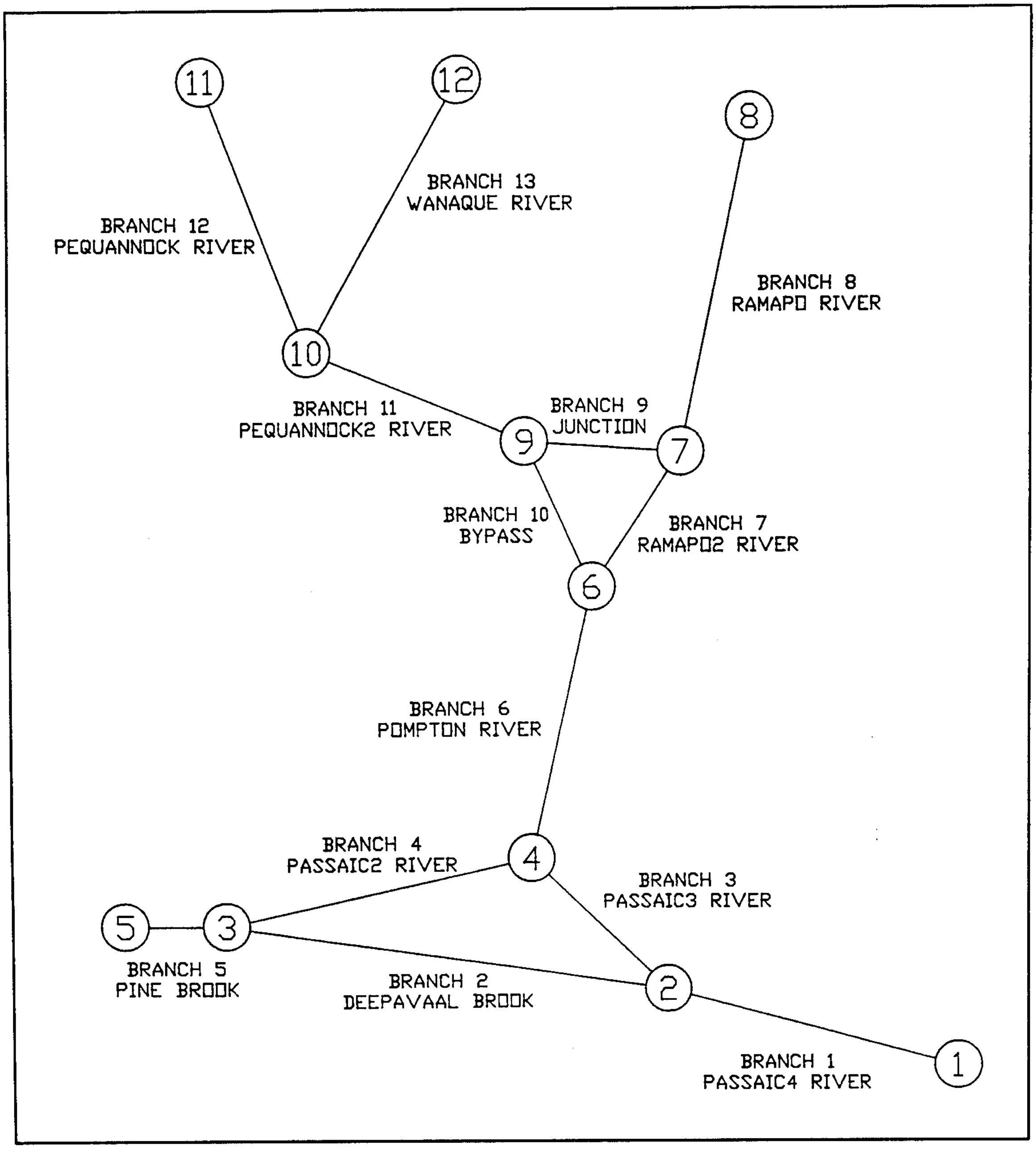

Figure 2. HEC-6 network schematic

2 Prototype Data

Geometric Data

The geometric data for the model were obtained almost exclusively from the UNET cross-section files. The first modification to the UNET geometry was the order of the cross sections in the file. This change was necessary because UNET computes from upstream to downstream (forewater) and HEC-6 computes from downstream to upstream (backwater). Other modifications to the geometry were the bridge sections. The bridge modifications required the deletion of closely spaced duplicate cross sections at bridges usually used by hydraulic models to transition from wider channels to the restricted bridge opening. Closely spaced cross sections should be avoided in HEC-6 since they are more sensitive to changes in sediment transport rates. A general rule of thumb for this model was to have a cross section approximately one bridge opening width upstream and a cross section four bridge opening widths downstream of the bridge cross section. This was the convention used in this model. The bridge cross sections themselves were modified to closely resemble actual sections. Other UNET cross sections at various locations in the model were modified so that they would more closely resemble the geometry observed from topographic maps and field observations. Finally, Manning's n values were generally used as in the UNET model, except in some locations when verifying the computed water-surface elevations of UNET and HEC-6 showed a different value was needed.

Sediment Data

Suspended sediment samples were collected by the USGS in May of 1992 and May of 1994. Both integrated and point samples were taken. The 1994 event was at a flow rate approximating a 1-year flood. Total sediment concentrations and the fractions of sand, silt, and clay were provided. These data showed concentrations from 2 to 20 mg/ℓ. The locations at which they were obtained were Pompton Plains, Singac, and Pine Brook. An analysis of the particle breakdown showed an average of 36 percent sand.

Bed material data were collected in May of 1992, May of 1994, and October of 1995. The samples were obtained manually using a punch

sampler, clamshell bottom sampler, and shovel. The gradation curves showed a D_{50} that varied from 130 mm in the Pequannock River to 0.2 mm in the Great Piece Meadows.

Hydrologic Data

Measured hydrologic data were extracted from the USGS sediment data files. Velocities, flow rates and water-surface elevations were obtained from the District from UNET simulations. These data files contained hourly discharges and water-surface elevations at each cross section in the model. The 2-, 25-, and 100-year flood frequencies were chosen as the storms to simulate, and were the ones from which data were extracted. The 100-year frequency represents the design event, the 25-year frequency approximates the April 1984 event (at Little Falls), and the 2-year frequency represents the channel-forming discharge and one of the smallest events for which the tunnel will be used. Several custom computer programs were written to extract the large quantities of these hydrologic data from the UNET files and place them in a format useable by HEC-6.

3 Field Observations of Existing Channels

Upper Reaches

At the time of the field survey, the upper reaches of the project, namely the Pequannock, Wanaque, and Ramapo Rivers, had, in general, fairly coarse bed material. The D_{50} ranged from 130 mm to 6 mm. The areas of greater channel slope contained coarser materials, and those of lesser slope contained the finer deposited material. The Wanaque River seemed stable except in the lower section near the mouth at Riverdale Road, and just downstream of the Paterson-Hamburg Turnpike Bridge. The slope of the river was steeper in this section than the overall channel slope. The water ran very clear, and no large areas of sand bars were noticed. The banks appeared stable in the observed areas of the reach, and no vegetation was noticed in the main channel. The upper Pequannock River had a varying slope, but was generally stable throughout its length. This seems to be due to the nearness of bedrock to the surface, and the coarseness of bed material. An area of relatively flat slope in the lower one-third of the reach appeared to have been an area of deposition in the past. Otherwise the channel characteristics resembled those of the Wanaque River.

The lower Pequannock River south of Riverdale Road had a constant overall slope except around station 46+00, where it was very flat and deposition was very evident. This flat section contained sand bars, deposited fines, and plant growth. Near station 29+65 the bed material became coarser within about 0.20 km (0.125 mile) of the weir, where fines were again more evident. Regarding bank stability, bank erosion scars were evident for some distance downstream of Riverdale Road. The Bypass channel (Branch 10) had a D_{50} of 16 mm and was rich in sand deposits and gravel. The banks seemed stable and were highly vegetated, the channel had minimal vegetation, and the slope was flat. The upper Ramapo River had a varying channel slope, being moderately steep in the upper section, and very flat, even adverse, in the lower section. There was no vegetation in the channel and moderate vegetation on the banks. Deposition was observed near the confluence with the Pequannock River. Other channel and hydraulic features resembled those of the lower Pequannock River. The lower Ramapo River in the vicinity of the proposed

tunnel inlet was an area of deposition due to the existing feeder dam at this location. Banks were highly vegetated, and the channel had moderate vegetation in some places. Bed materials were sands and fines, and velocities were less than 0.152 m/sec (0.5 ft/sec) at low flows.

Pompton River

The Pompton River is about 11.26 km (7 miles) in length. The channel slope varied, with the steepest section being in the upper reaches. Here, the bottom material could be relatively coarse, D_{50} of 40 mm, with no vegetation in the channel and generally stable banks. Some bedrock appeared upstream of the Pompton Turnpike. In the lower two-fifths of the reach there were areas of deposition and much channel sinuosity. The banks were unstable, as indicated by the local residents' attempts at stabilizing their property. The bed gradations contained more fines as near Two Bridges, with a D_{50} of 0.75 mm. However, at Two Bridges, the bed gradation became very coarse again with a D_{50} of 35 mm. Apparently this was the result of eddies and reverse flows during large floods at the confluence of the two rivers. Taken as a whole, the Pompton River seemed to be the most active reach of the network with regard to scour, deposition, and bank stability.

Passaic River

The Passaic River upstream of Two Bridges, the apparent zone of influence of the tunnel project for the 100-year flood, extends upstream to the vicinity of Pine Brook. From Pine Brook to Two Bridges, the bed gradations were fine, with the channel being very sinuous. Here the river passes through the Great Piece Meadows, which is a wetland. There was generally no vegetation in the channel, but heavy vegetation in the overbanks during the growing season. In the vicinity of Two Bridges, the bed gradations became very coarse, for the same apparent reasons as in the Pompton River at this location. Downstream of Two Bridges and upstream of Interstate 80, there was a zone of deposition. This could be due to the initial delta development caused by the backwater influence of Beatties Dam. Further downstream the portions of river that were observed seemed to be stable. There were rock outcrops in the areas of the dams, and the few bed gradation samples had D_{50}'s ranging from 4 mm to 65 mm.

Deepavaal Brook

This reach has a varying slope, being steeper in the upper sections. At its upper end, it receives overflow from the Passaic River during very large events. Bed samples taken at the lower end had a D_{50} of 16 mm. It was not observed in detail with regard to vegetation and channel stability.

4 Base Test

Geometric Model

The geometric model is based on the network schematic as shown in Figure 2. It consists of a file of cross sections arranged sequentially from downstream to upstream. Thirteen reaches in all were used. Four hundred and six cross sections were needed. In this file, n values for flow resistance, moveable bed limits, moveable bed depth, channel limits, and conveyance limits were also specified. The values of the different parameters were selected based on field observation, prior UNET files where applicable, information gleaned from topographic maps, and experience. The dams and weirs were modeled by setting the height of the dam as the channel bottom and preventing it from scouring. With respect to cross-sectional area, bridges were modeled using a special feature of HEC-6 that subtracts the areas of piers and embankments from the cross-sectional area.

Sedimentary Model

The portion of the model that moves sediment requires specifying a transport function, inflowing concentration versus flow data, and bed gradations. The hydraulic analysis package SAM[1] was used to assist in selecting the most suitable transport function. Based on the factors of channel slope, velocity, width, depth, and D_{50}, the Yang function was used. The concentration versus flow graphs were developed using field data collected by the USGS in the spring of 1994. Information was available from the Pine Brook, Singac, and Pompton Plains gauging sites. This information is used by the model to specify inflowing concentrations at model boundaries and local inflows for varying flow rates. Finally, information about the channel bed material properties is required by the model. With the exception of the Wolman count done on the Pequannock River, all of the bed gradations were

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[1] W. A. Thomas, R. R. Copeland, N. K. Raphelt, and D. N. McComas. "User's manual for the Hydraulic Design Package for Channels (SAM)" (in preparation), U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

entered into the model according to the locations from which they were taken.

Hydrologic Model

Flow and stage hydrographs produced by the UNET unsteady flow model were obtained from the District. The flood frequencies were the 2-year, 25-year, and 100-year events. The stage data were used mainly in the verification of the HEC-6 hydraulic output. The flow data were used as direct input to the HEC-6 model. The hydrographs for all 406 cross sections in the network were obtained from the UNET run at 6-hour time-steps. A program was written that arranged the order of the input file to coincide with the order in which HEC-6 processes each cross section. So, for a given time-step, the flow rate for each cross section was listed in its sequential order. The same procedure was used for the next time-step and so on. For each flood frequency there were about 56 time-steps, representing a hydrograph of about 14 days duration. By using the available hydrologic data in this way, both the spatial and temporal integrity of the various flood waves moving through the network was preserved in the HEC-6 model runs. This was especially important in areas of reverse flow and at times where phased peaks occurred. Throughout the simulation no correction was made to the flows for changes in bed geometry.

One other very important aspect of the hydrology of the model was the channel-forming discharge concept. In short, this concept proposes that the major features of channel geometry throughout the life of a river or stream are brought about by the floods that run near bank-full. Floods of this dimension have generally been found to be associated with 2-year flood frequencies. For this reason it was considered very important to simulate the 2-year flood. This was done with an eye on simulating the changes that might occur to the channel over a longer period of time. The model results of these changes were not meant to be definitive in terms of absolute magnitude, but only in the sense of defining trends of deposition or scour. The changes to the network over 50 years were important to assess. In order to simulate such changes, historical hydraulic data were required. Such data as needed for the HEC-6 model were not available. There were historical data, but not at all the locations and over the time periods required. In the absence of such data, the channel-forming discharge concept was invoked. From 100 years of historical records from the District, the average daily flow was 32.5 cu m/sec (1,148 cu ft/sec) at Little Falls. This translates to about 1.019 billion cubic meters (36 billion cubic feet) of water flowing through the channel in a year's time. The peak flow of the 2-year flood is approximately 147.8 cu m/sec (5,220 cu ft/sec) at Little Falls after diversion. One would need to run that flow rate through the channel continuously for about 80 days in order to move the same volume of water that is moved in a year's time by the average daily flow. In order to simulate 50 years of volume using the 2-year flood, it would be necessary to run a simulation for 80 days per year for 50 years, or a 4,000-day simulation.

Verification

When the model was complete in all aspects–geometry, sediment, and hydrologic data–the initial runs were made. The year 2000 was arbitrarily used to represent time equal to zero, or the initial condition. The initial condition runs were made with a single time-step of 6 hours so that no significant bed changes were allowed to occur. In this way it was assured that the HEC-6 bed profile matched that of the UNET runs, and thus a valid comparison could be made between the water-surface elevations calculated by HEC-6 and those calculated by UNET. The 2-year peak flow in the area of the Junction Branch and occurring on day 3, hour 2400, was selected as the flow against which to verify. The Junction Branch is at the confluence of the Pequannock and Ramapo Rivers. The purpose of these verification runs was to check the hydraulic performance of HEC-6. With the proper hydrologic input, HEC-6 should reproduce correct water-surface elevations. In the absence of correctly calculated hydraulic parameters, the subsequent sediment calculations would be meaningless. After some minor adjustments to Manning's n values, the water-surface profiles of the UNET runs were matched very closely. Figure 3 shows the comparison of HEC-6 and UNET water-surface profiles for Branch 6, the Pompton River. The match was adequate to assure correctly calculated hydraulic parameters. In Appendixes A and B the comparison graphs of any branch in the network are presented. Overall, the comparisons were more than adequate. The legends of the graphs in both the figures and appendices show from left to right: the flood frequency, the year of the simulation, water-surface or channel bed, base or plan, and specific notes. For example, the legend 2-2000 WS PLN HEC-6 indicates that the graph is for a 2-year flood in the year 2000 and shows the water-surface elevations for the plan condition for an HEC-6 simulation.

50-Year Simulation (Base)

The District requested that WES investigate both the base (unimproved) and plan (improved) conditions 50 years (year 2050) after the initial condition (year 2000). The methodology used to accomplish this task is discussed under the section, "Hydrologic Model." The same 2-year peak flow used in the model verification was used in running the model through its 50-year simulation of total volume of flow. Again, the approach was to view the model results in terms of trends, not in an absolute sense of differences of bed change magnitudes. Figure 4 shows the initial condition (year 2000) bed and water surface compared to the future condition 50 years later (2050) for the Pompton River. In general the trend is toward degradation of the channel from the upstream reaches down to about section 7000. Below this section there are several areas of expected deposition. A corresponding drop in the water surface can be seen in the upper reaches. Please refer to Appendix C for the graphical displays that support the following discussion. In Branch 3 a worrisome trend showing deposition in the area between sections 172000 and 173000 was observed. This happens to be the area where the tunnel spur inlet

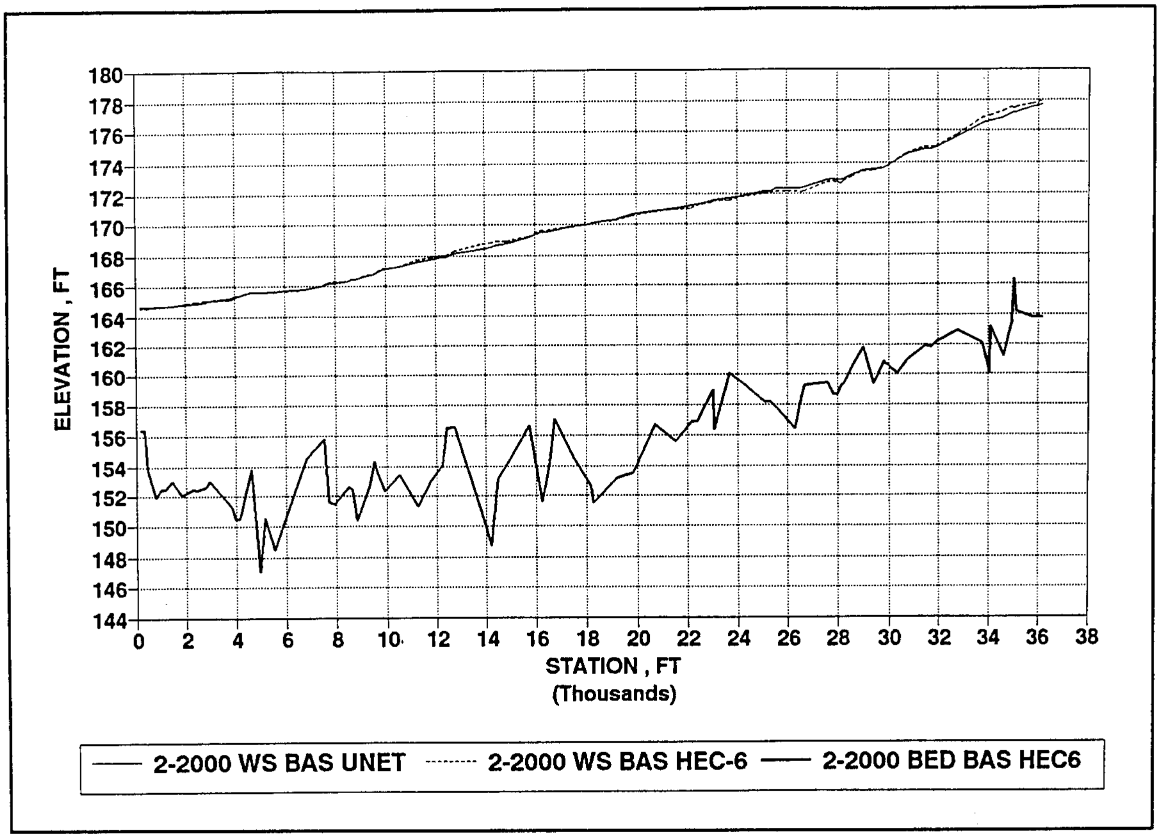

Figure 3. Bed and water-surface profiles, Branch 6, Pompton River, year 2000. Note: To convert elevations and stationing given in feet to meters, multiply by 0.3048.

will be located. There was also excessive deposition on Branch 4 in the vicinity of Two Bridges. Later runs and further investigation showed that indeed, these are areas that tend toward deposition. However, the magnitudes shown in these plots are exaggerated due to the steady state nature of the channel-forming discharge. It must be remembered that this channel-forming discharge run was made as a steady state run, and the time chosen happened to have reverse flow at this location. The reverse flow had a lower velocity and thus allowed continual deposition at this location. When a full hydrograph is run through the system, the series of depositing reverse flows followed by forward flushing flows produces the more realistic bed changes of about one meter. The other area of major concern is the Pompton tunnel inlet (main inlet). The upper reaches of the Pompton River show bed degradation, as does the lower reach of the Bypass channel. Branch 11, the lower Pequannock River, also shows a trend toward degradation except around sections 2700 and 5000.

Overall there were no major concerns in the base runs, and the verification of hydraulic parameters was adequate.

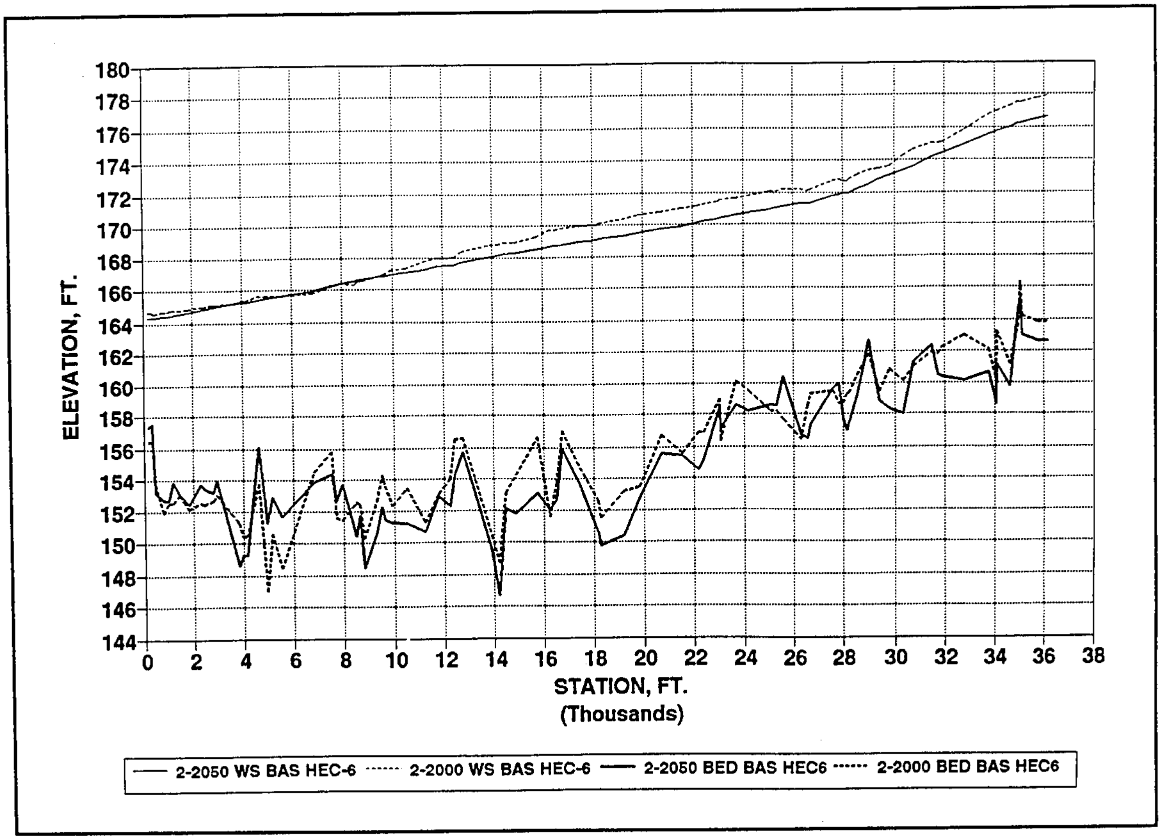

Figure 4. Bed and water-surface profiles, Branch 6, Pompton River, 50-year simulation of the base. Note: To convert elevations and stationing given in feet to meters, multiply by 0.3048.

5 Plan Test

Geometric Model

The geometry file of the base test was modified to produce the plan geometry file. The UNET geometry files for the plan condition were obtained from District personnel, along with maps and drawings. For channel improvements and levees, the UNET plan cross sections were inserted in the base file in place of the existing cross sections. Channel improvements were made on the Pequannock, Wanaque, Ramapo, Bypass, Pompton, and Passaic Rivers, as well as on Deepavaal Brook. Levees were added on the Ramapo and lower Pequannock Rivers and the Passaic River near Pine Brook. A pilot channel consisting of a trapezoidal notch in the bottom of the cross section to convey low flows was added in the Passaic River downstream of the spur inlet for about 1.60 km (1 mile). The structures requiring coding modification were the Great Piece Weir and the new Bypass/Pequannock Weir. The Great Piece Weir will not be operated until after the peak flows have passed and rain has ceased, so the fully open invert position was input as a hard point. The Bypass/Pequannock Weir will maintain a constant pool elevation of 54.22 m (177.9 ft).[1] The HEC-6 code was modified to keep the water-surface elevation of section 1500 on the Bypass Branch at that elevation throughout the plan simulations. Finally, the two tunnel inlets were modeled as local outflows.

Sedimentary Model

The only changes made to the sediment input section of the model were those concerning the fraction of sediment that might pass into the tunnel inlets. With a one-dimensional model, only an estimate can be made. This was done using information provided by an HEC-6 run with the Toffaleti transport function activated. With an E-level printout turned on, a detailed listing of the sediment grain sizes, their transport rate in tons per day, and their approximate vertical location in the water column was obtained. Knowing the river invert

[1] All elevations (el) cited herein are in meters (feet) referred to the National Geodetic Vertical Datum (NGVD).

at the tunnel entrance and the tunnel inlet crest elevation, it is possible to calculate what percentage of the different grain sizes in the water column can pass over the crest of the tunnel inlet using the data from Toffaleti's listing. These percentages were entered as coefficients in the input deck. If a zero is entered, no sediment is transported to the flow diversion (tunnel inlet in this case). If a 1 is entered, 100 percent of the sediment transported in the tunnel's portion of flow is moved into the tunnel. In reality, most grain sizes will have coefficients less than 1, with the larger grain sizes having smaller coefficients. These coefficients were calculated for the sand and gravel transported in the model for the 2-year and 100-year flood for both the main and spur tunnel inlets. The values are shown in the following tabulation:

Location	Grain Size Coefficient			
	Very Fine Sand	Fine Sand	Medium Sand	Coarse Sand
Pompton Inlet	0.83	0.79	0.53	0.09
Spur Inlet	0.73	0.42	0.07	0

The coefficient values are only very crude approximations, being obtained from a one-dimensional model, and should not be considered definitive. The actual transport phenomenon will be governed by flow streamlines and secondary currents, neither of which is accounted for by any one-dimensional model. At present, a physical model must be used to obtain more accurate and definitive information regarding sediment transported into the tunnel.

Hydrologic Model

As with the base condition, the District provided UNET values of stage and flows for all cross sections in 14-day hydrographs. The data were then extracted and put into the HEC-6 format. Other changes that were required were the operating schedules of the tunnel inlets and the overflow weir on the Passaic River into Deepavaal Brook. The tunnel inlets were modeled as local outflows and Deepavaal Brook as a local inflow. The flow hydrographs were modified accordingly.

Verification

With all the plan (improved condition) changes in place, the initial runs were made. Verification of the HEC-6 plan runs took place using the same procedures as in the base condition by checking the model hydraulic performance. The only difference was that the "plan" geometry, sediment properties, and hydrology were in the model. The water-surface profiles of the UNET model were matched by HEC-6 very closely with two notable exceptions. The first was on Branch 2, Deepavaal Brook, where the two profiles diverge by

about 0.3048 to 0.4572 m (1 to 1-1/2 ft) after station 14+500. This is shown in Figure 5. Two likely reasons for this are flood routing speed and the low-flow pilot channel used in the UNET model. The other notable exception where the profiles do not match is at cross section 1500 of Branch 10, the Bypass Channel. This is shown in Figure 6. The reason for this mismatch is that the weir in this location is modeled as section 1150 in the HEC-6 model, and as section 1780 in the UNET model. The HEC-6 code was modified to hold the water surface of section 1500 at el 54.22 (177.9) throughout the simulation. The pool elevation was being maintained at that level during any flood event. The District estimates that only at floods larger than the 100-year event might the pool rise higher than that elevation. The profiles of the downstream end of the lower Pequannock River again match well, attesting to hydraulic continuity between the Bypass Channel and the lower Pequannock River branches. This profile as well as all others for the plan verification runs can be seen in Appendix B. Overall, the comparisons were favorable and more than adequate to assure validity of hydraulic parameters.

50-Year Simulation (Plan)

As in the base condition, an analysis of the plan condition 50 years

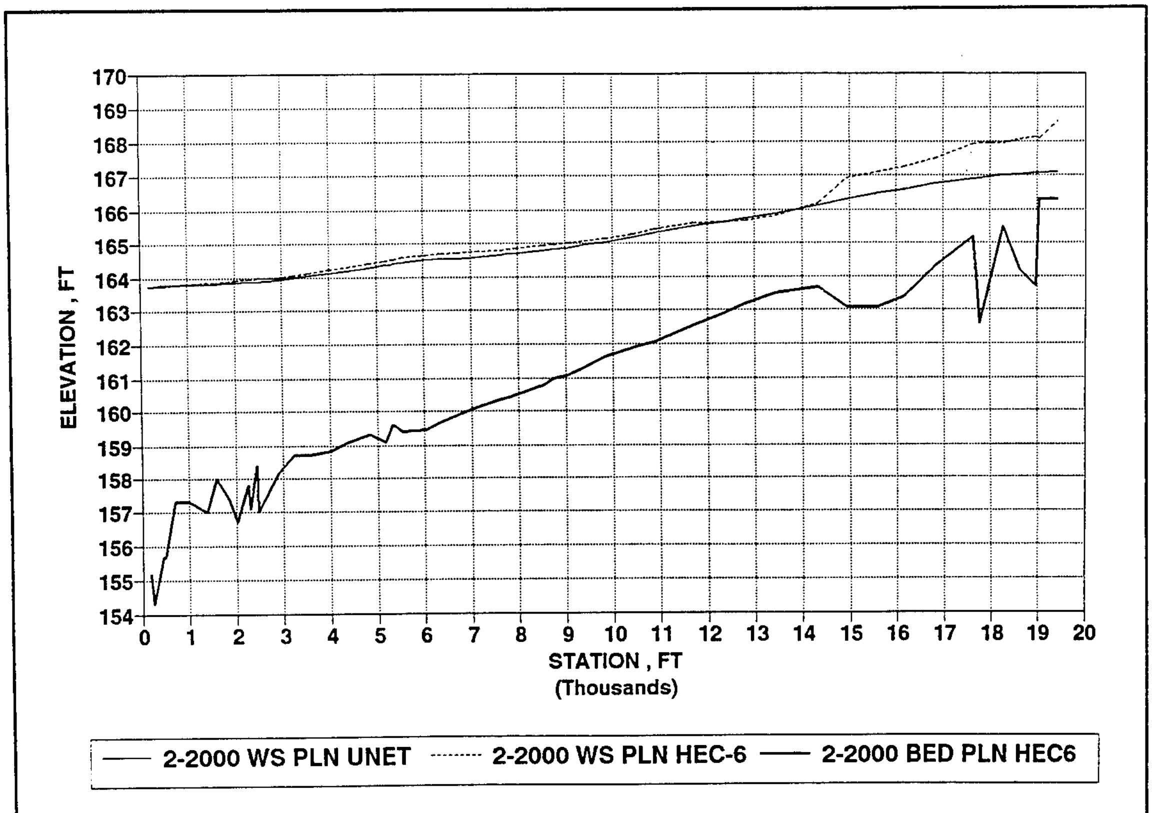

Figure 5. Bed and water-surface profiles, Branch 2, Deepavaal Brook. Note: To convert elevations and stationing given in feet to meters, multiply by 0.3048.

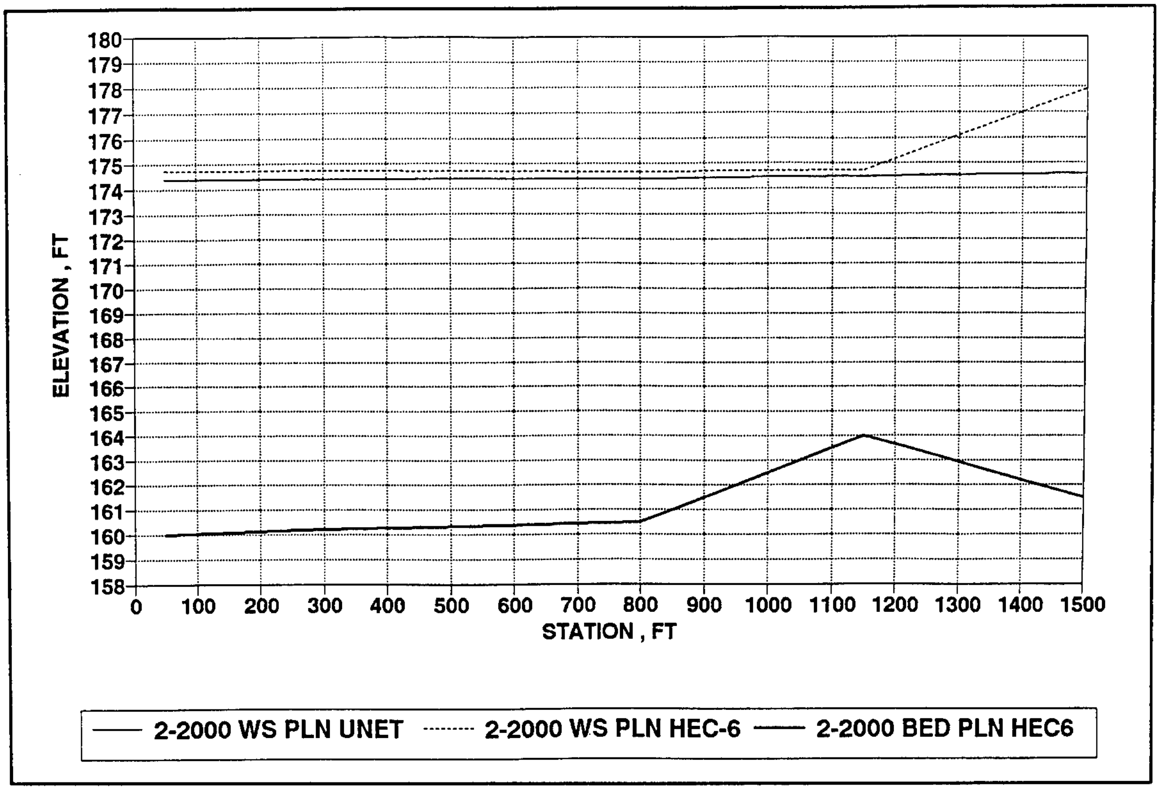

Figure 6. Bed and water-surface profiles, Branch 10, Bypass Channel. Note: To convert elevations and stationing given in feet to meters, multiply by 0.3048.

(year 2050) after the initial condition (year 2000) was conducted. The methodology is discussed under the section, "Hydrologic Model," in Chapter 4. The same 2-year peak flow used in the model verification was used in running the model through its 50-year total volume simulation. Once again, the approach was to view the model results in terms of trends, not in an absolute sense of differences of bed change magnitudes. Figure 7 shows the year 2000 bed of the Pompton River compared to the year 2050 bed. As in the base condition, a trend toward degradation of the channel is evident except in isolated locations and at the downstream end of the channel near Two Bridges. There was a corresponding drop in the water surface to match the bed change. Near the spur inlet on Branch 3, there was also a trend toward deposition, as in the base condition. There appears to be no trend to deposit in the vicinity of the Pompton inlet. How the other branches reacted to the 50-year simulation can be viewed in Appendix D.

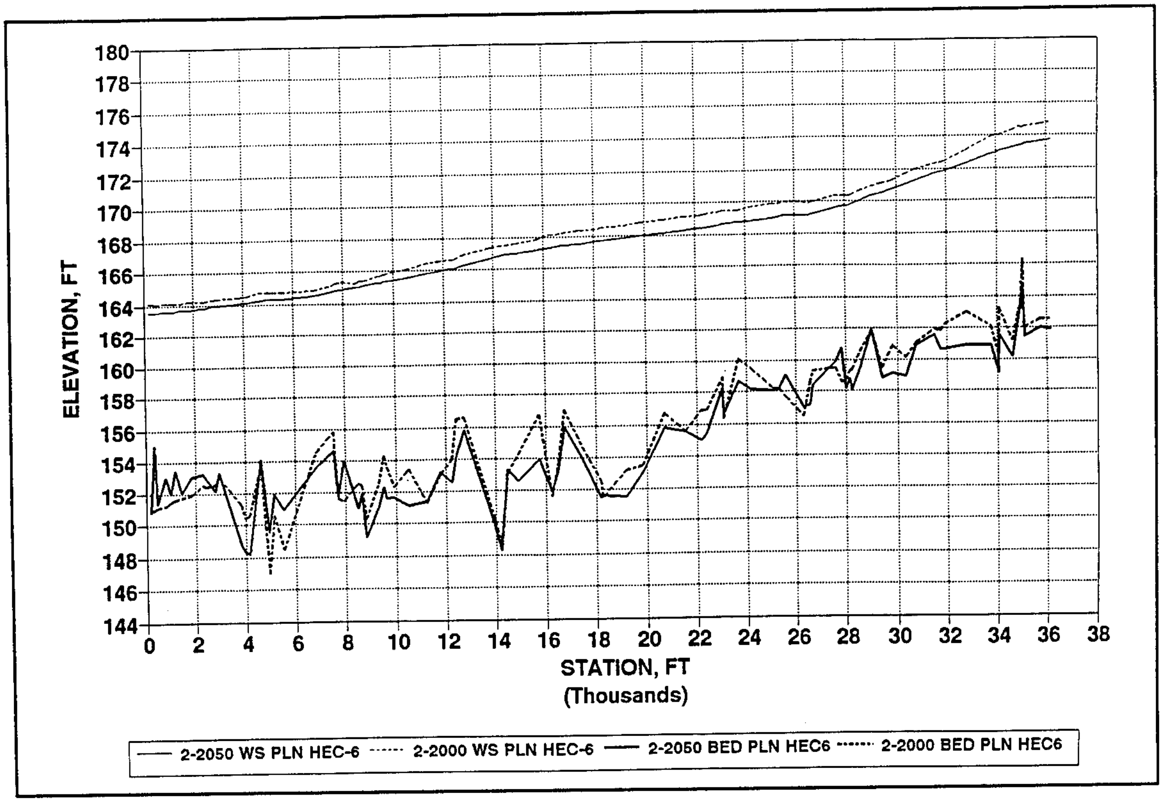

Figure 7. Bed and water-surface profiles, Branch 6, Pompton River, 50-year simulation plan. Note: To convert elevations and stationing given in feet to meters, multiply by 0.3048.

6 Base and Plan Test Results

Initial Condition

Appendix E shows graphs of the base and plan together for the year 2000 simulation. As noted before, these results are based on a run duration of one time-step and the bed geometry was not given time to change. Thus, any differences in water-surface profiles were due only to changes in geometry and/or hydrology of base versus plan, and not due to time. As can be seen in most of the graphs, the water surfaces in the areas of channel improvements were lower for the plan. Also, the Pompton River water-surface profile was significantly lowered, not due to channel improvements, but due to the tunnel. From this perspective, the plan seemed to achieve the desired results.

Year 2050

Appendix F shows graphs of the base and plan together for the year 2050 simulation. This series of graphs adds the dimension of time to the prior set of graphs in Appendix E. They are steady state runs and show how the base and plan differ with regard to long-term trends. The dynamic impacts of short-term flood hydrographs are presented in the following section, "Hydrographs." Figure 8 shows the Pompton River. Notice that over the 50 years, the base condition tended to degrade the bed more than the plan did. Other areas, notably Branch 3, showed similar tendencies.

Hydrographs

The 2-, 25-, and 100-year hydrographs were superimposed on the results of the base and plan year 2050 runs. This adds the dynamic impacts of short-term floods to the long-term results discussed in the previous paragraph. In each series of graphs the maximum water surface attained was plotted. In addition the year 2000 (initial condition) results were plotted. In this case the initial condition runs were not just one time-step, but a 14-day hydrograph of

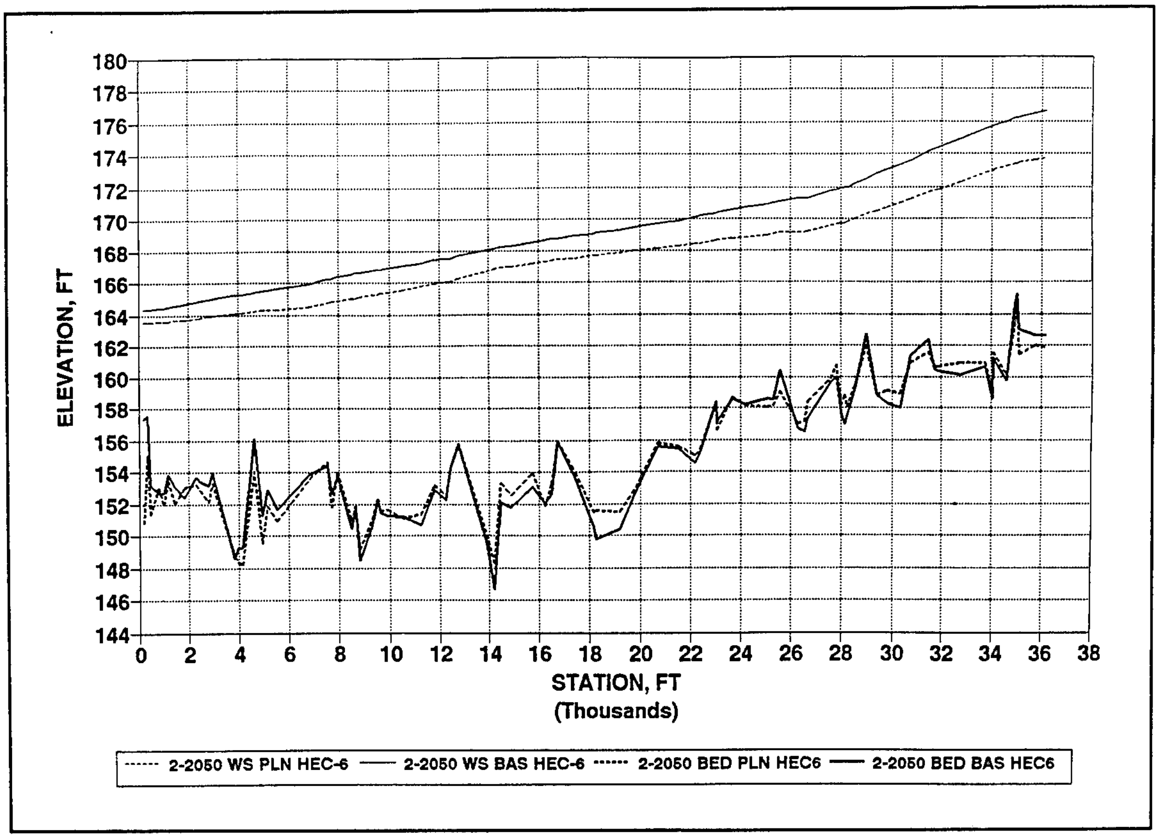

Figure 8. Bed and water-surface profiles, Branch 6, Pompton River, comparison of base and plan, year 2050. Note: To convert elevations and stationing given in feet to meters, multiply by 0.3048.

the given flood frequency. This allows one to see the differences between base and plan for the various flood frequencies, as well as how the plan fared after 50 years of operation. There were no major surprises. These graphs can be viewed in Appendices G, H and I for the 2-, 25-, and 100-year hydrographs, respectively. Rather than discuss all the details in these many graphs, the general conclusions drawn from them relevant to this study are summarized in Chapter 7.

7 Conclusions and Recommendations

With the plan in place, reductions in water surfaces were attained in most branches of the upstream network. The greatest reductions were for the 100-year flood frequency. In general, differences between the plan at year 2000 and year 2050 were relatively small. This indicated that the plan itself is not self-defeating at any noticeable location. In other words, the plan in general is stable once it is in place and in operation over 50 years and through the various flood hydrographs. Thus there is no indication that excessive long-term maintenance will be required.

The sedimentation characteristics of the network require a bit more discussion. The main places of concern were Two Bridges, the Great Piece Weir, the tunnel inlets, and the Pompton River. At Two Bridges there was much concern with the amount of deposition noticed during field observations and from the model long-term runs, especially in view of the fact that the Great Piece Weir was proposed for this vicinity. It was concluded from field observations and the model runs that in the vicinity of Two Bridges, deposition of up to 0.914 m (3 ft) might occur for the 100-year flood. This could indicate a necessity for short-term maintenance after extreme events. The proposed location of the Great Piece Weir 213 m (700 ft) upstream of Two Bridges appears to put the weir just at the edge of this deposition zone. Upstream of the Great Piece Weir there was minimal deposition, probably fines supplied from the wetland.

The spur inlet of the tunnel was just downstream of section 172310 of Branch 3. Without exception every run showed this area as a zone of deposition. As previously mentioned, it could be due to the initial delta development caused by the backwater influence of Beatties Dam. The amount of deposition at this location seemed to be reduced by the plan, possibly as a result of the reduced degradation in the Pompton River. To obtain a gross estimate of the amount of sediment that might enter the tunnel, a separate trap efficiency study was made.[1]

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[1] Philip W. Thomas and William A. Thomas. (1995). "Sedimentation in the proposed tunnel, Passaic River Flood Protection Project," letter report submitted to Hydraulics Laboratory, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

The Pompton River appeared to be the most unstable reach in the network. It showed degradation in the upper sections and some deposition in the lower reaches. The rate of change at either location seemed to be reduced by the plan. Initially it was thought that the increased water slope due to the tunnel operation would increase the tendency of degradation in this river. However, the 100-year plan runs compared to the 100-year base runs showed that not only were the water-surface elevations lower in the plan, but the slope of the water surface was less. This accounted for the reduced rates of degradation. However, it was recommended that since this area was already known to be unstable, the District photo-document the streambanks to show that such was the case at the beginning of the project. If not, future degradation could be attributed to the project, when in fact the opposite appeared to be true. The project seemed to reduce the rate of degradation due to flows equal to or exceeding the 2-year flood.

At the upper end of the Pompton River in the vicinity of the main inlet, there was a very slight indication toward scour. In Branch 10, which adjoins Branch 6 in the same vicinity but upstream of the main inlet, a tendency for deposition seemed evident. Due to the channel improvements at this location, some deposition was expected as the system tried to return to equilibrium. As with the spur inlet, defining the percentages and distribution of sediments that would actually enter the tunnel was beyond the capabilities of the HEC-6 model and should be addressed in the recommended physical model. The physical model would help to determine the types and amount of materials that would accumulate directly in front of the tunnel inlets, and thus help define the maintenance requirements to keep the sediment traps and skimming structures operating effectively. No simulation was made for the 500-year flood. Such an event could possibly carry large material into the tunnel that could not be flushed out and would therefore require maintenance for removal. The physical model would also help to quantify the sizes of materials that could possibly be moved into the tunnel in extreme events.

The last area to address was section 2165 near the Pequannock Weir, which adjoins section 1500 of the Bypass Branch. The plan runs showed that over 50 years, some scour might occur just upstream of the weir. This was not expected since deposition normally occurs behind dams. However, a check was made of the model sediment concentrations in this location for the 2-year flood. These were compared with measured sediment concentration taken in March of 1994 in the vicinity of Pompton Plains. The measured values averaged 5.2 mg/ℓ. From HEC-6 the base run sediment concentration from day 3, hour 1800, was 5.4 mg/ℓ, and for the plan run on day 3 at hour 0600, 2.2 mg/ℓ. Therefore the model agreed well with measured data. With such low concentrations, it was possible that some scour could occur in this area, even though the trend to do so appeared to be minimal. Due to the uncertainty caused by this model result it was suggested to monitor the channel cross

section at this location with sedimentation ranges as in EM 1110-2-4000,[1] Appendix K, paragraphs 6 to 17. Although reservoirs are explicitly described, the procedures can easily be used for rivers and streams. It was highly recommended that the tunnel inlets and Great Piece Weir also be monitored in such a manner.

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[1] Headquarters, U.S. Army Corps of Engineers. (1989 (15 December), Change 1, 31 October 1995). "Sedimentation investigations of rivers and reservoirs," EM 1110-2-4000, Washington, DC.

Appendix A
Base Condition Verification

This appendix contains graphs for river branches 1 to 13 showing the HEC-6 and UNET water-surface elevation verification plots for the base condition using a 2-year steady state peak flow for a single time-step. To convert feet to meters, multiply by 0.3048.

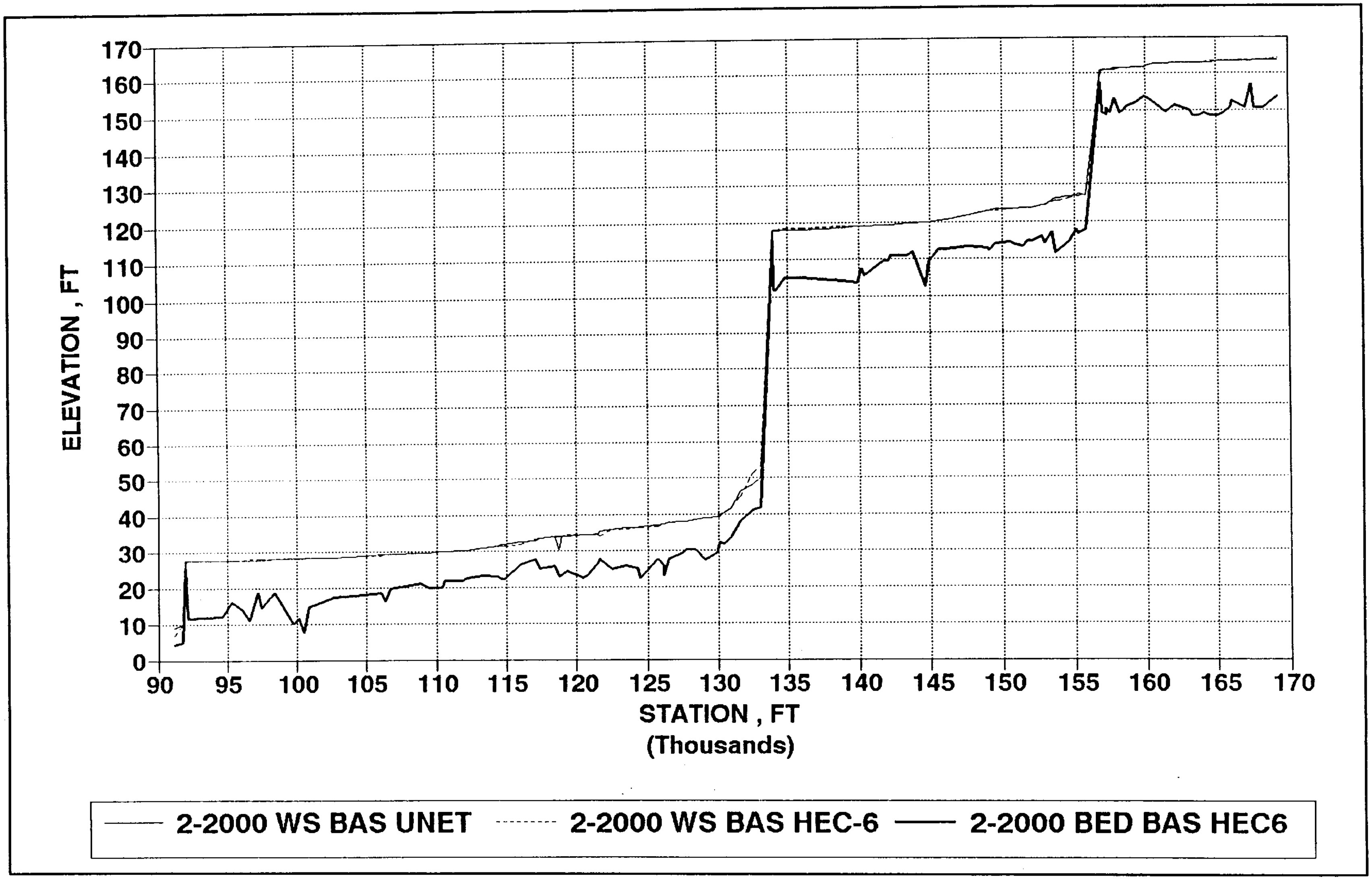


Figure A1. Base condition verification, bed and water-surface profiles, Branch 1, Passaic River

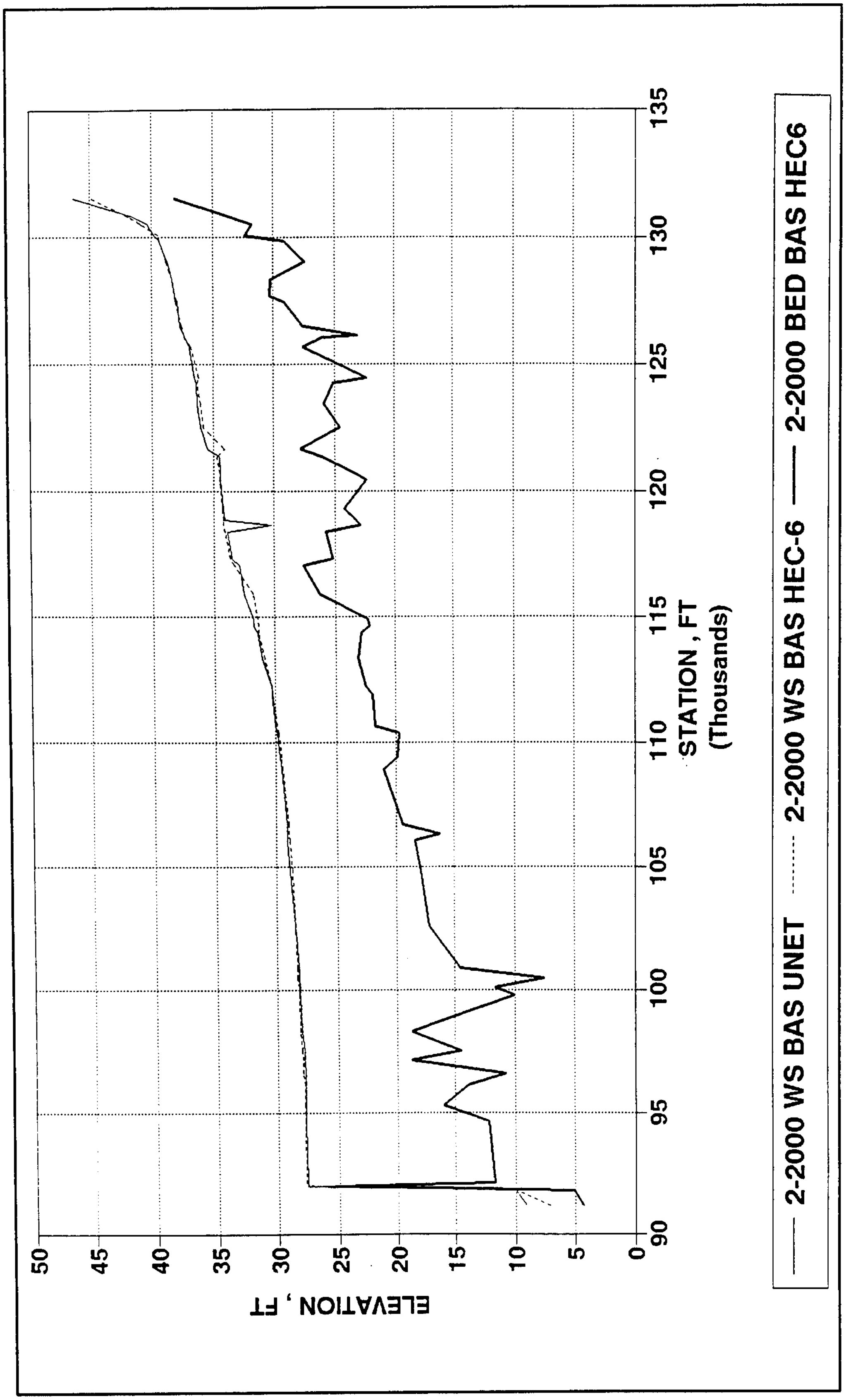


Figure A2. Base condition verification, bed and water-surface profiles, Branch 1, Passaic River, Dundee Dam

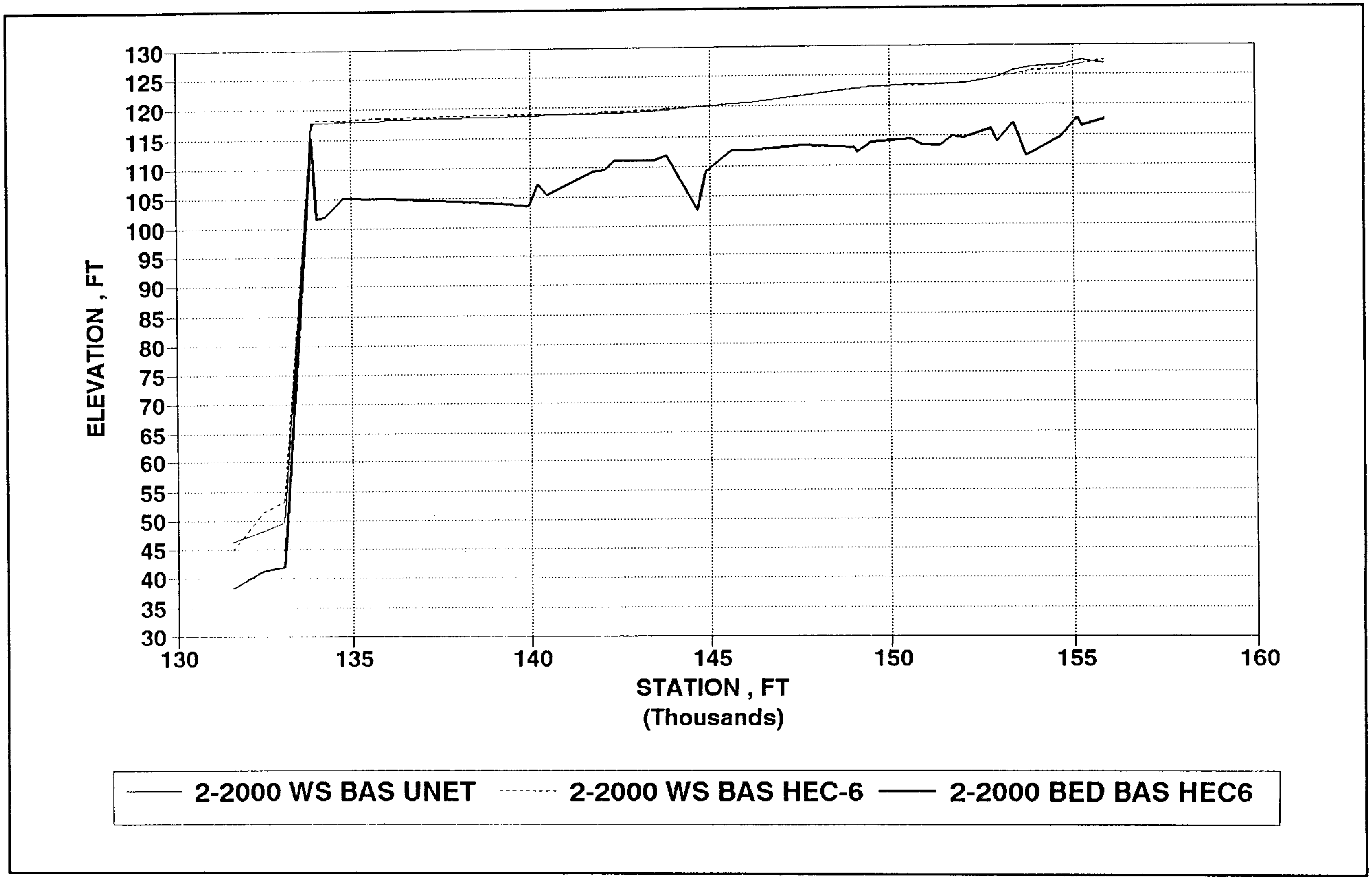


Figure A3. Base condition verification, bed and water-surface profiles, Branch 1, Passaic River, S.U.M. Dam

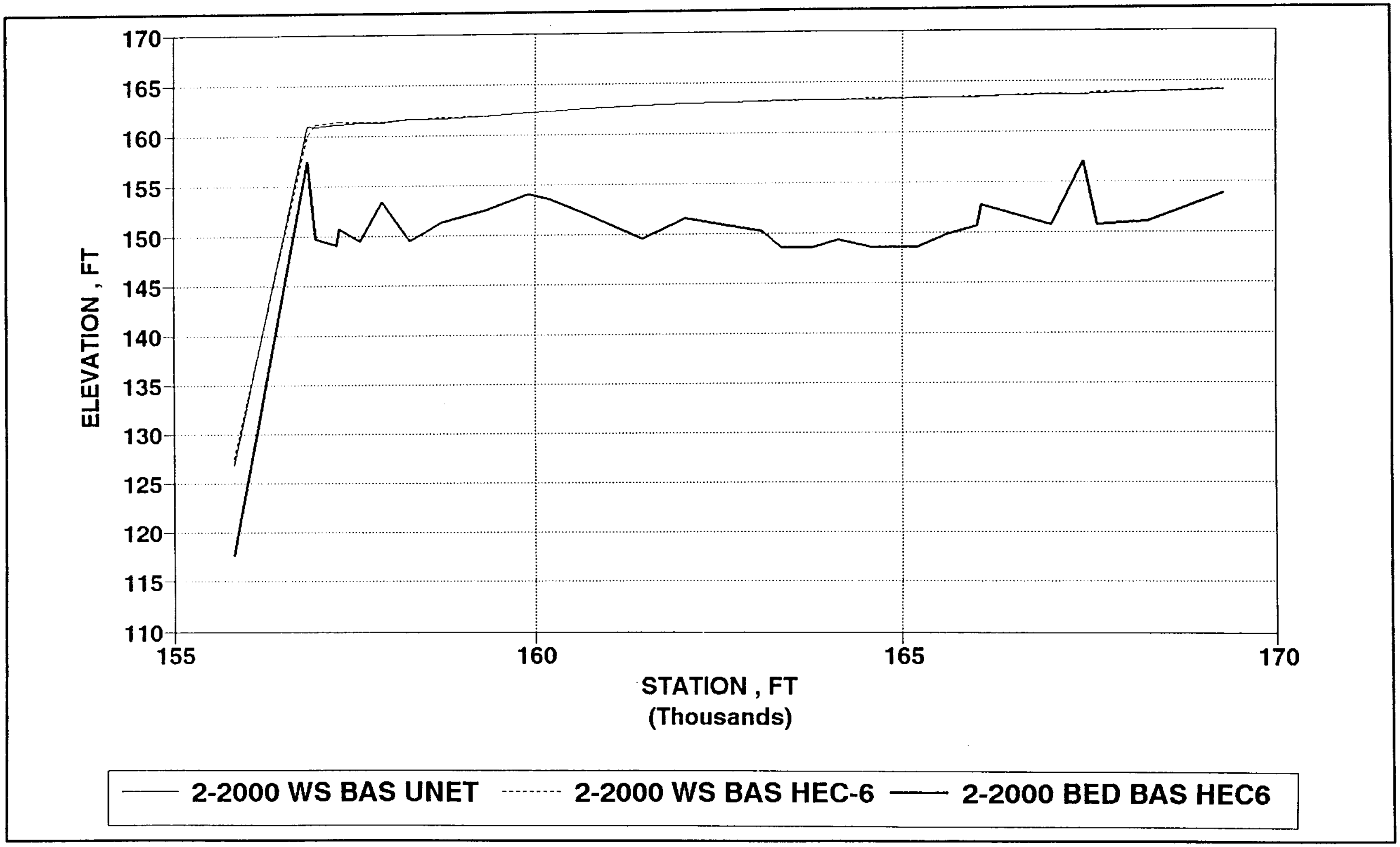


Figure A4. Base condition verification, bed and water-surface profiles, Branch 1, Passaic River, Beatties Dam

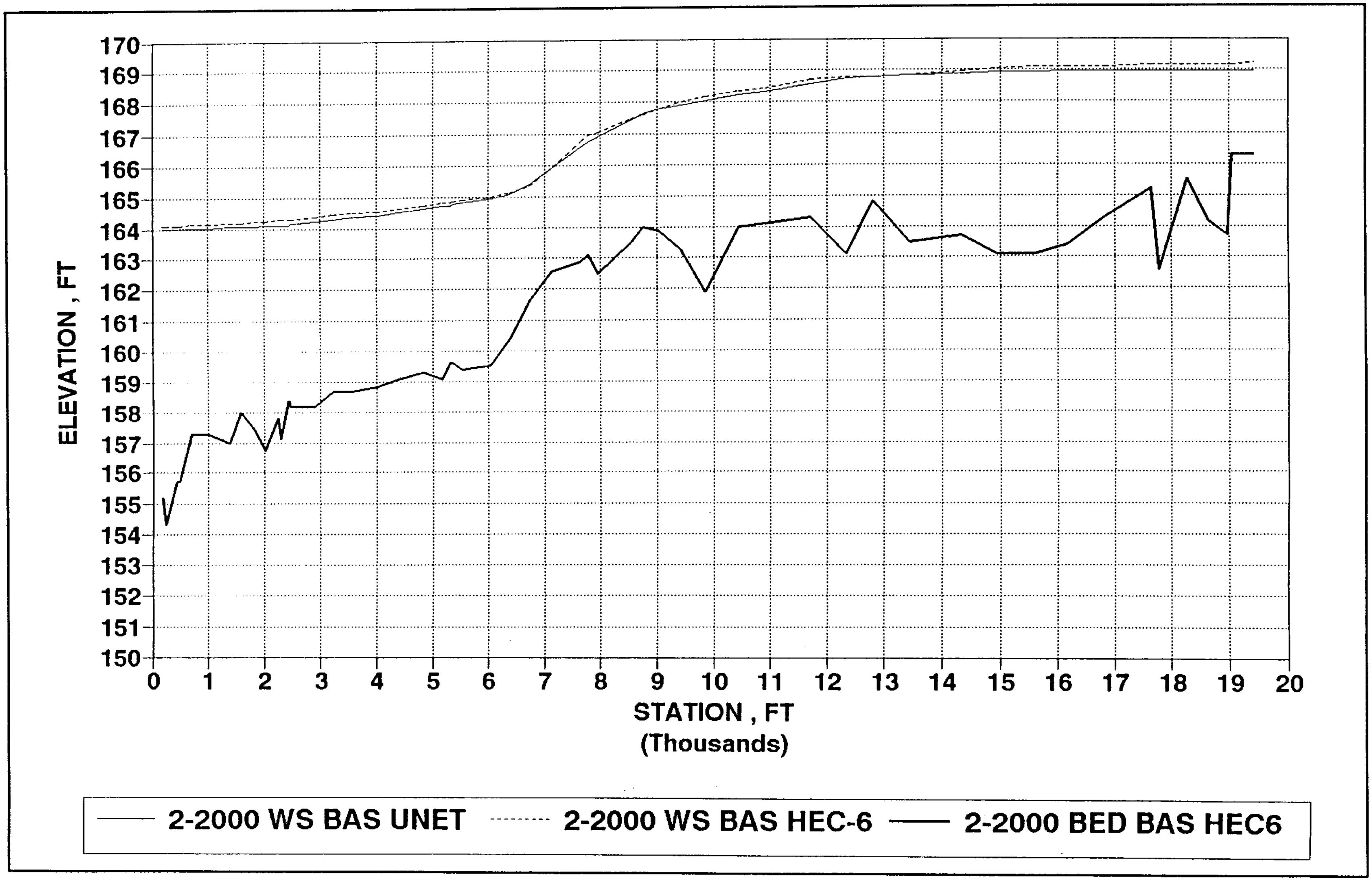


Figure A5. Base condition verification, bed and water-surface profiles, Branch 2, Deepavaal Brook

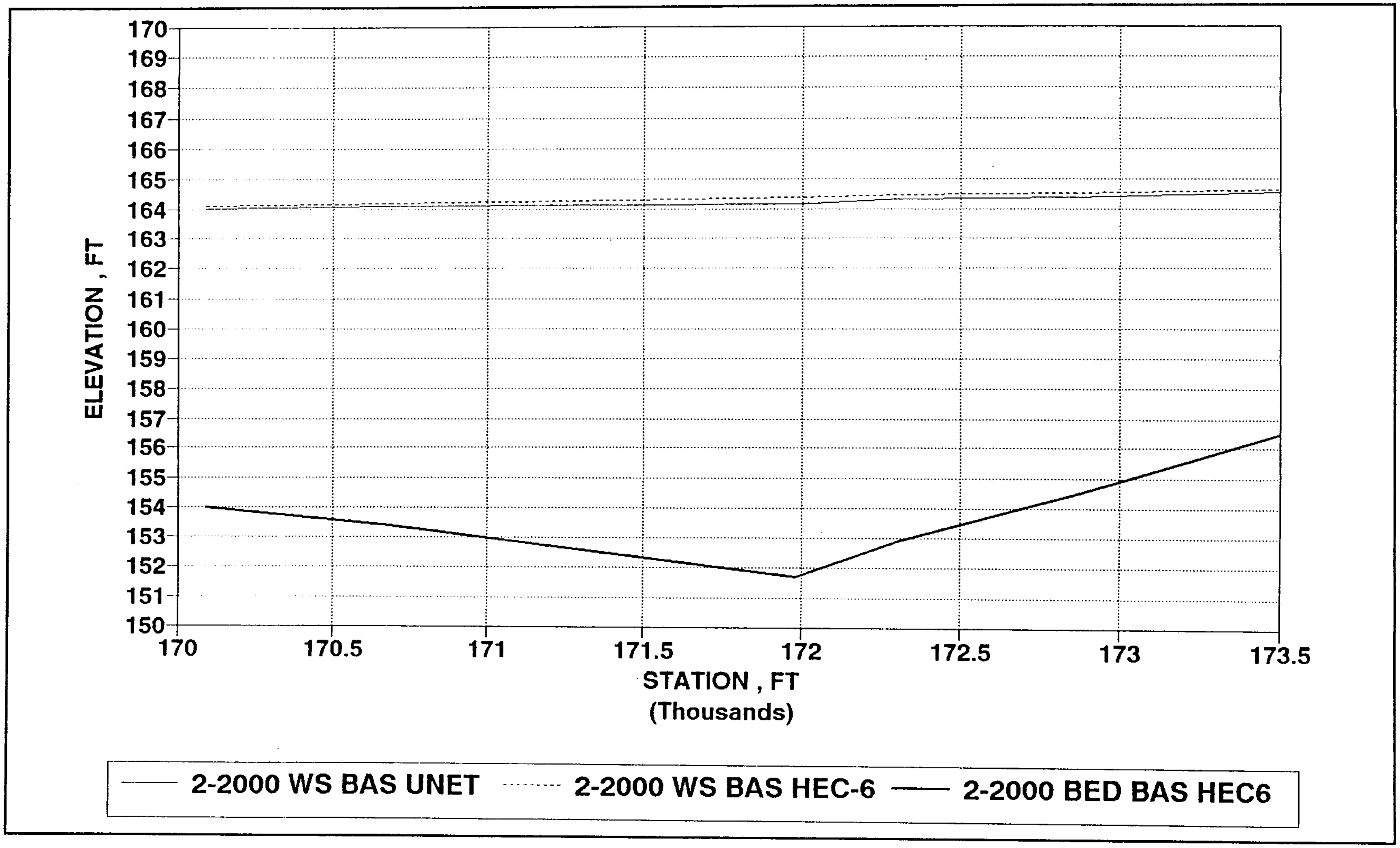


Figure A6. Base condition verification, bed and water-surface profiles, Branch 3, Passaic River

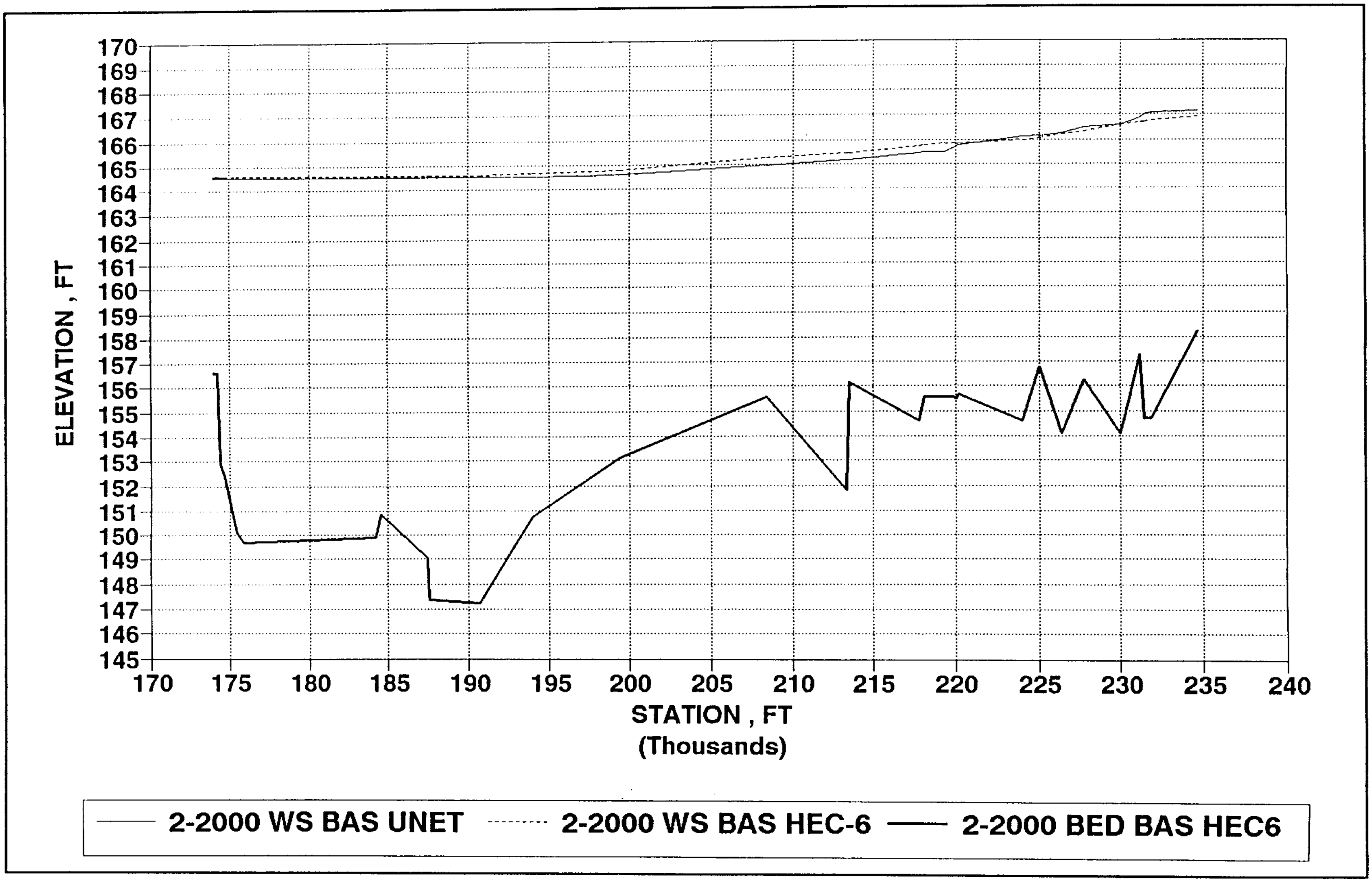


Figure A7. Base condition verification, bed and water-surface profiles, Branch 4, Passaic River

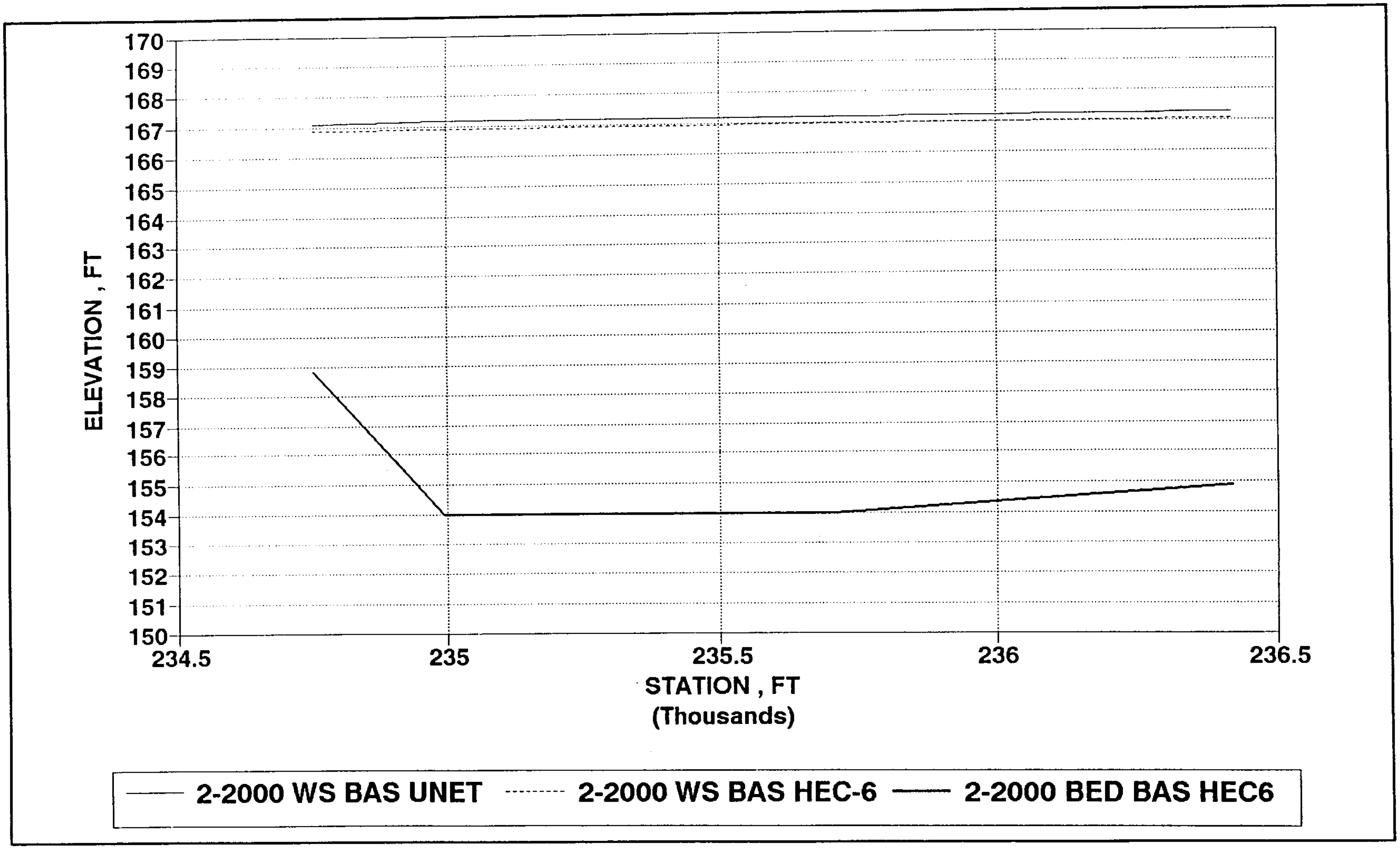


Figure A8. Base condition verification, bed and water-surface profiles, Branch 5, Passaic River

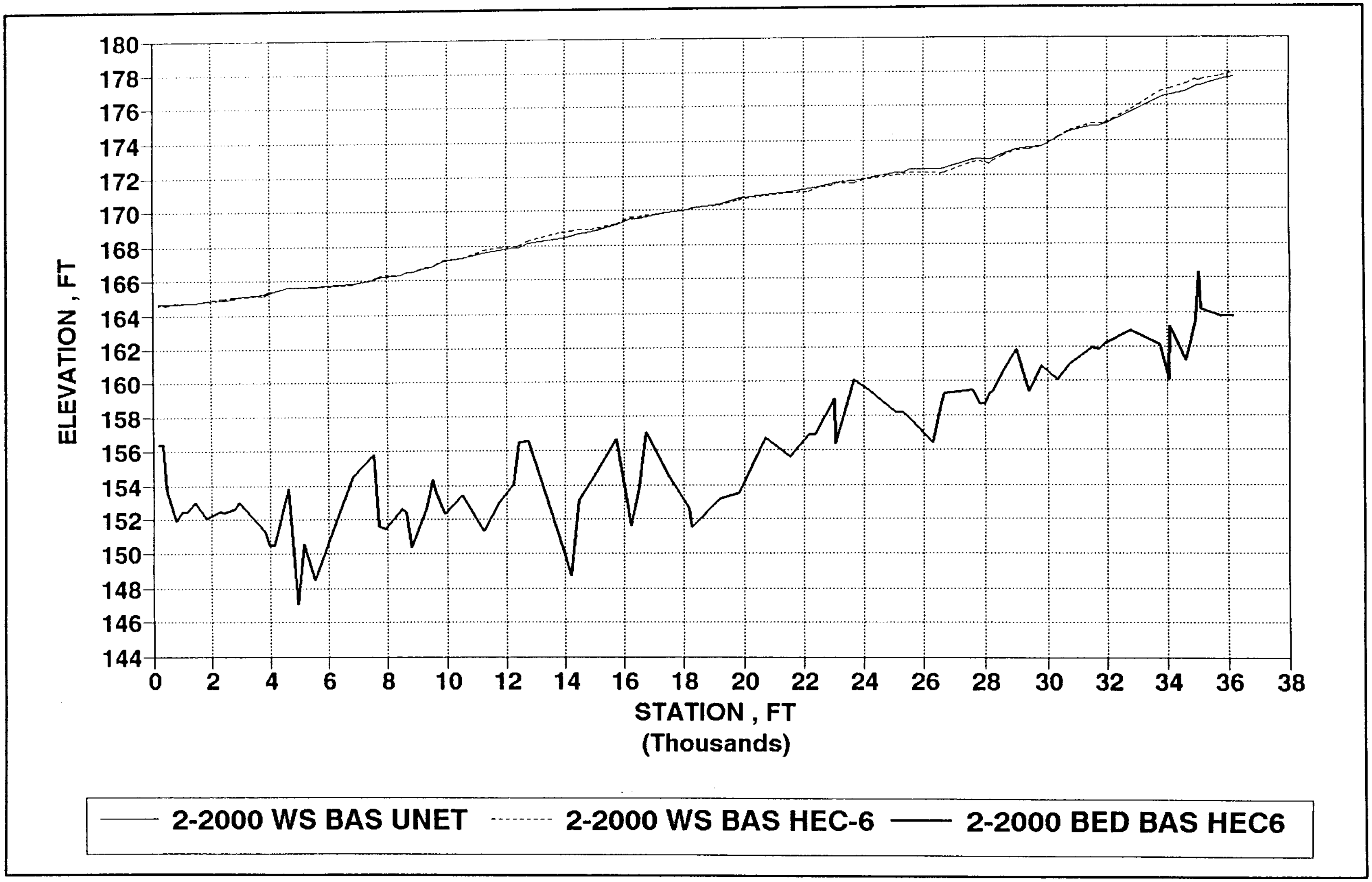


Figure A9. Base condition verification, bed and water-surface profiles, Branch 6, Pompton River

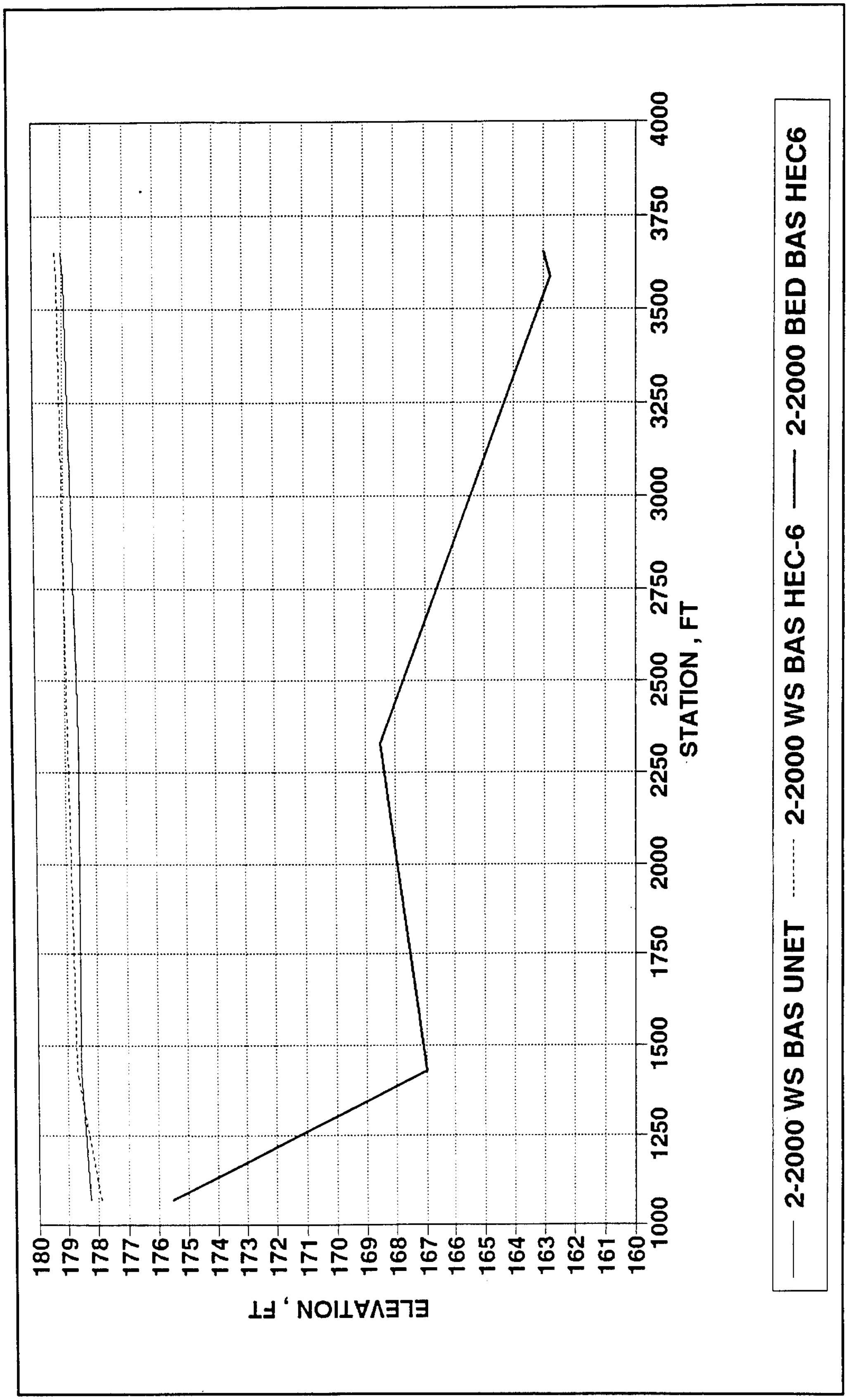


Figure A10. Base condition verification, bed and water-surface profiles, Branch 7, Ramapo River

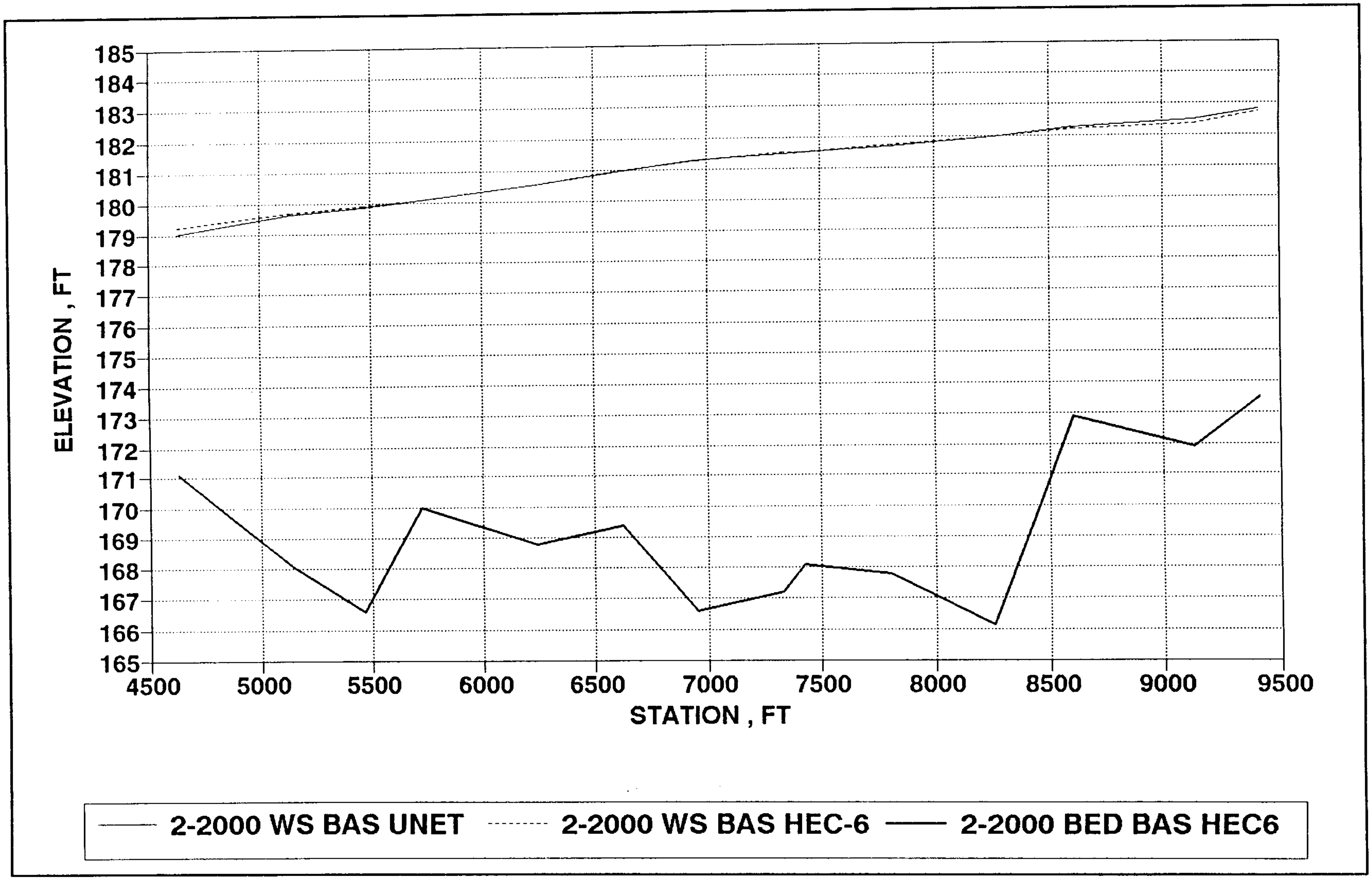


Figure A11. Base condition verification, bed and water-surface profiles, Branch 8, Ramapo River

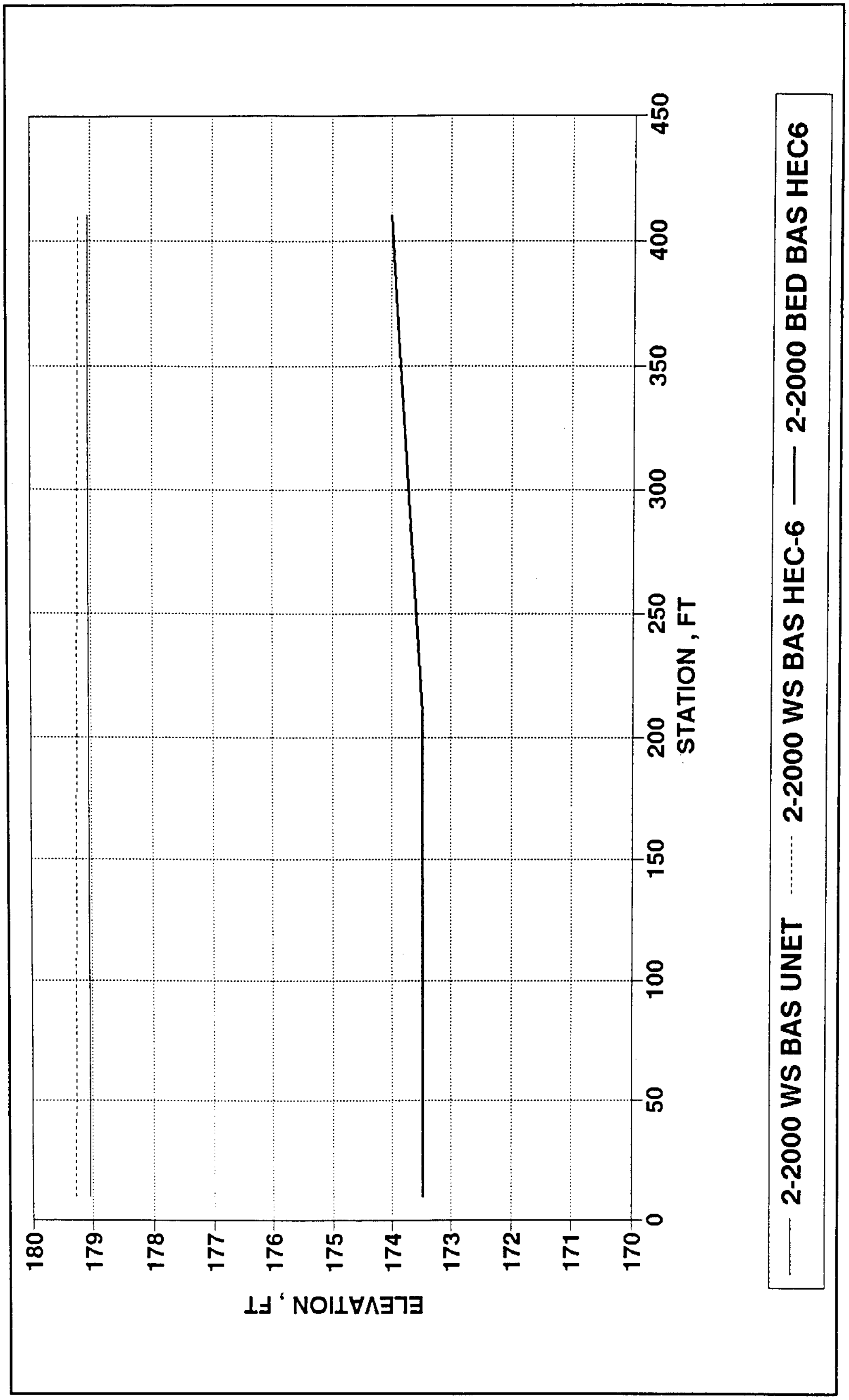


Figure A12. Base condition verification, bed and water-surface profiles, Branch 9

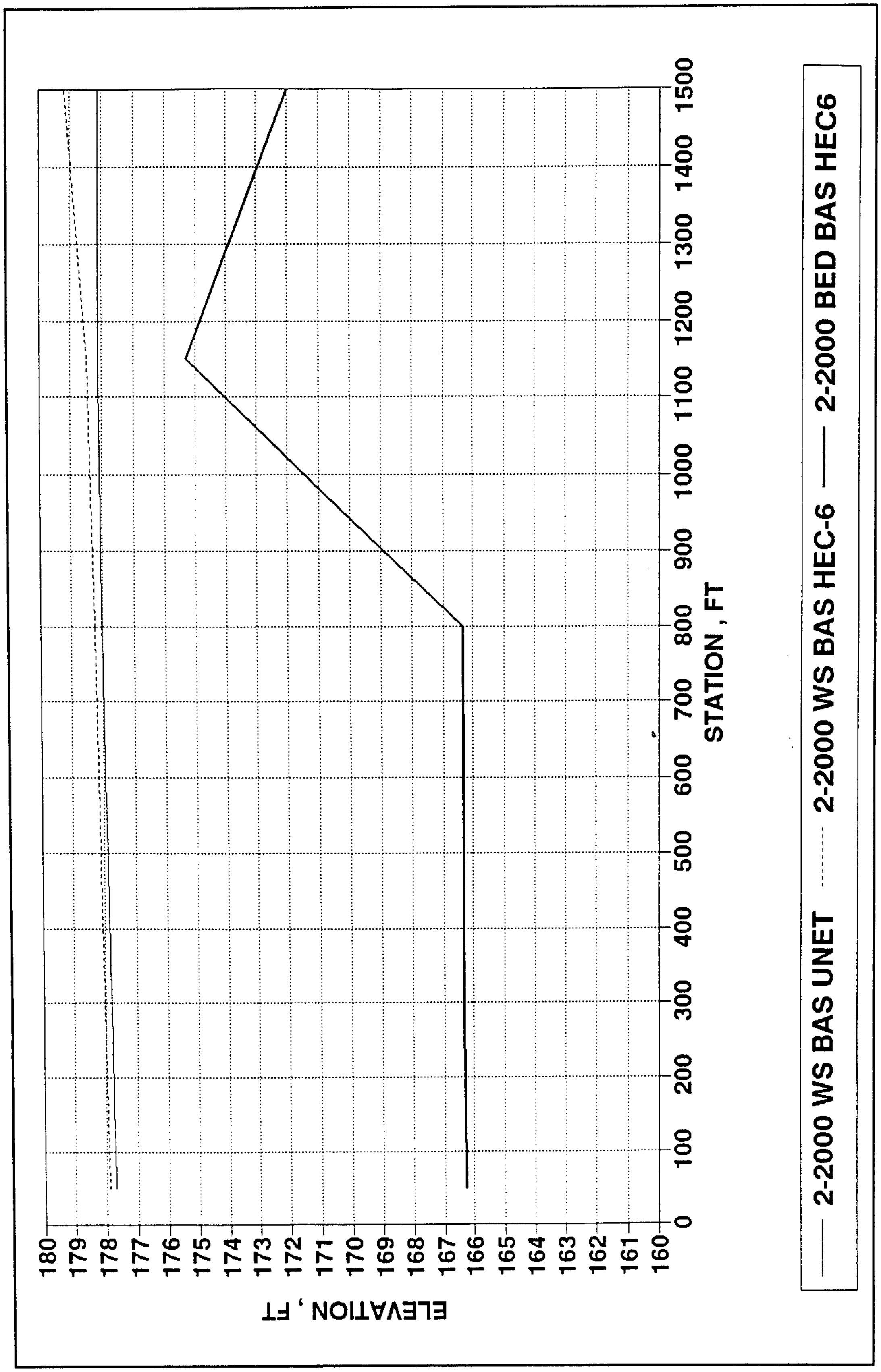


Figure A13. Base condition verification, bed and water-surface profiles, Branch 10, Bypass

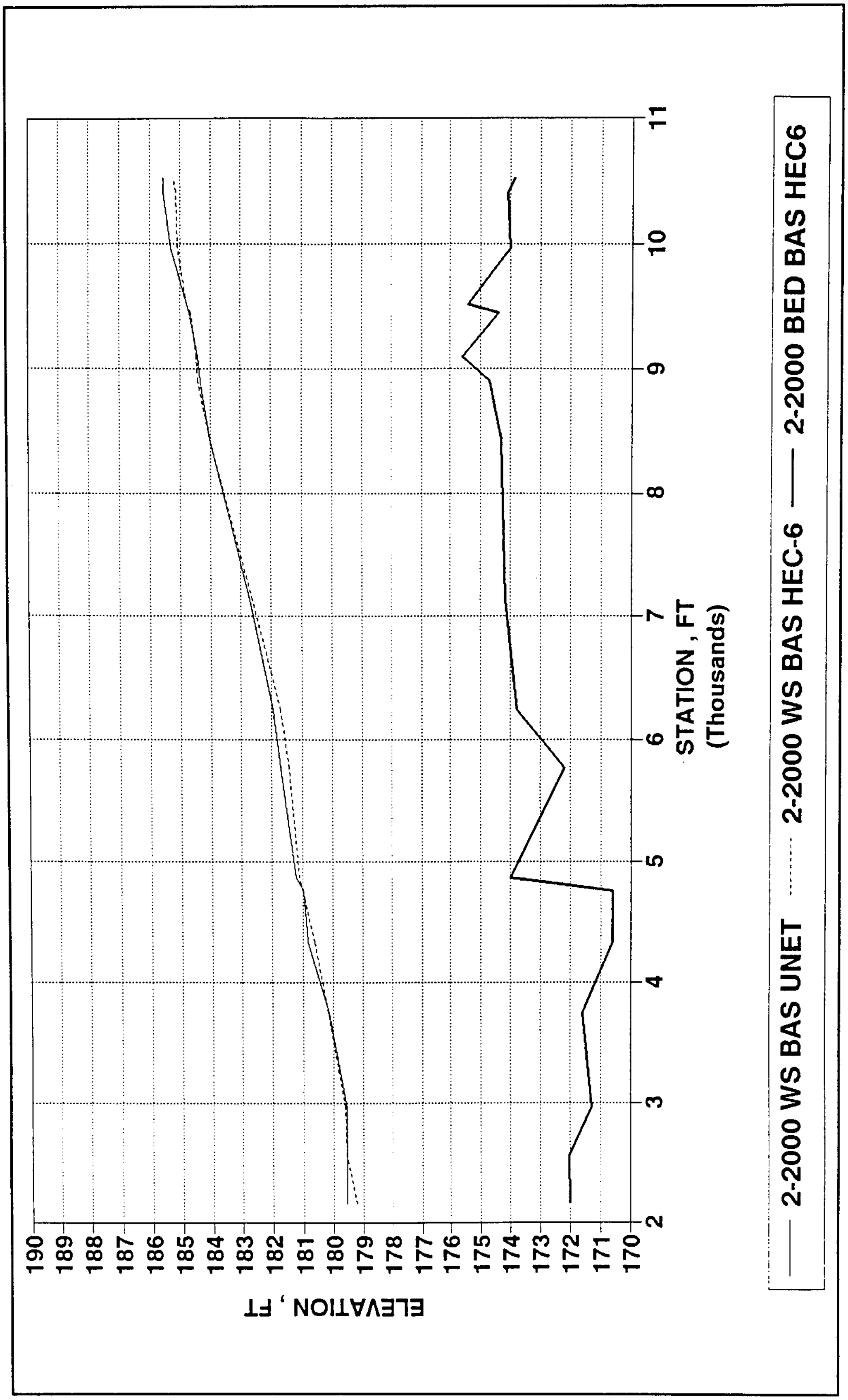


Figure A14. Base condition verification, bed and water-surface profiles, Branch 11, Pequannock River

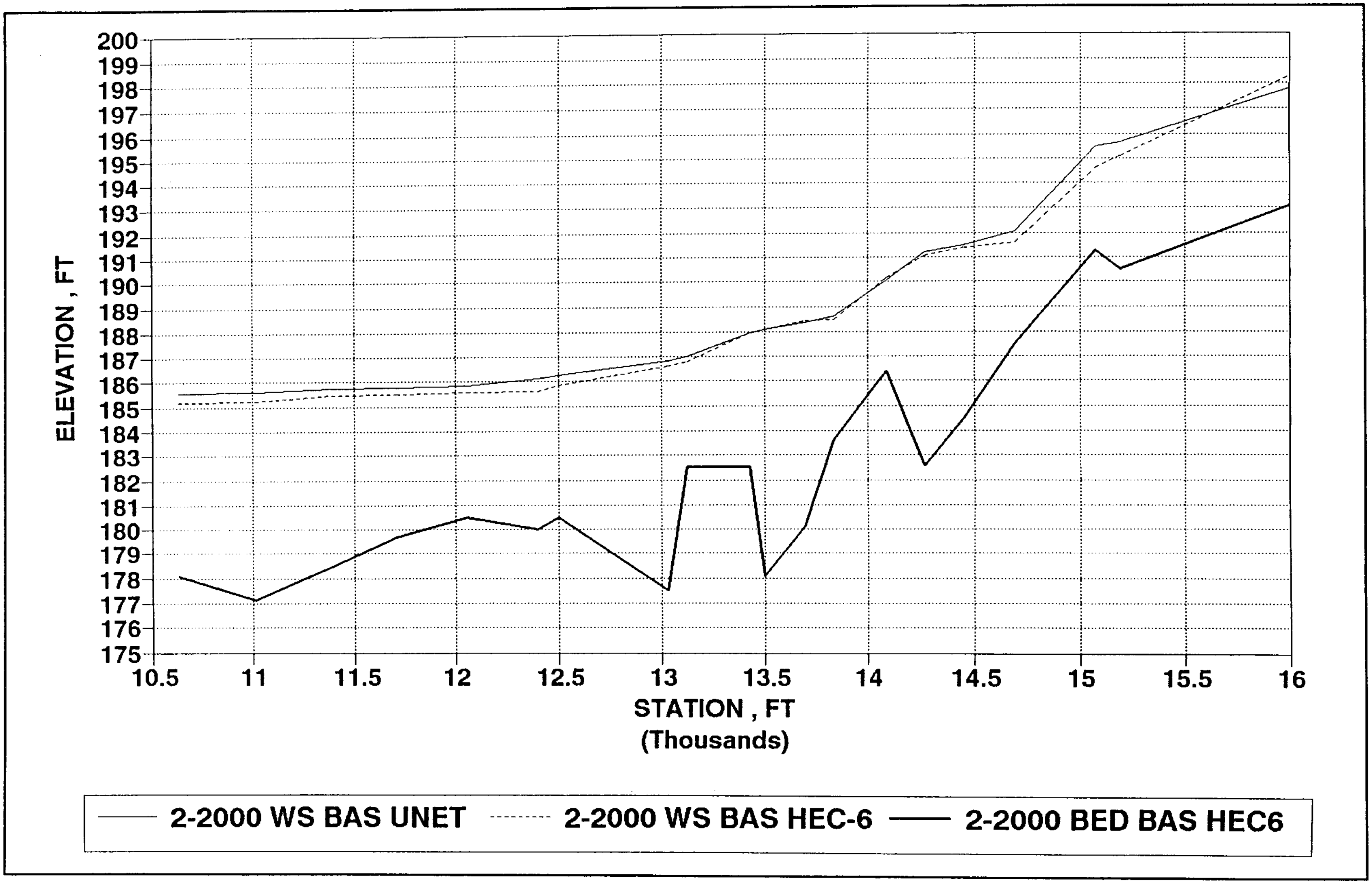


Figure A15. Base condition verification, bed and water-surface profiles, Branch 12, Pequannock River

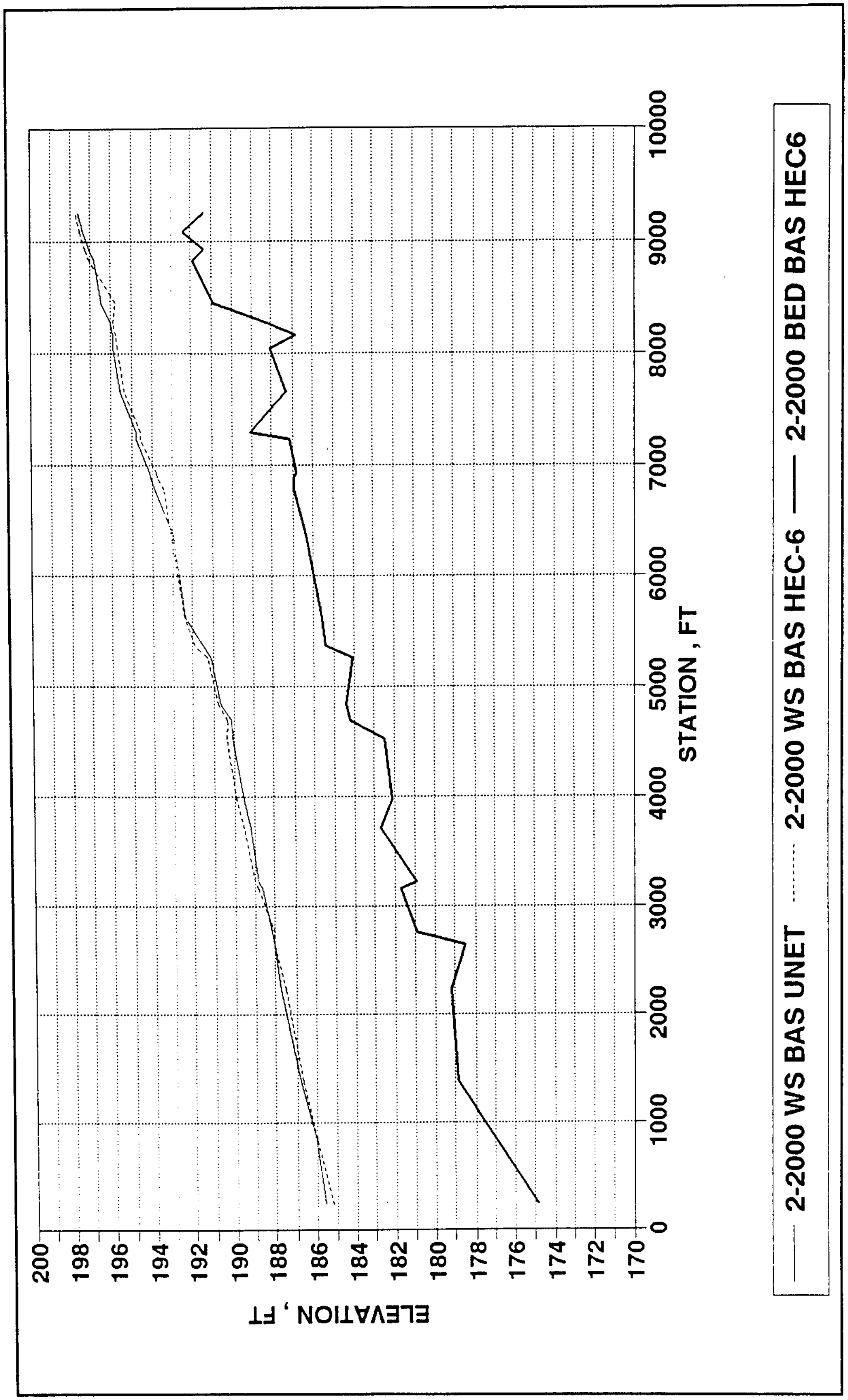


Figure A16. Base condition verification, bed and water-surface profiles, Branch 13, Wanaque River

Appendix B
Plan Condition Verification

This appendix contains graphs for river branches 1 to 13 showing the HEC-6 and UNET water-surface elevation verification plots for the plan condition using a 2-year steady state peak flow for a single time-step. To convert feet to meters, multiply by 0.3048.

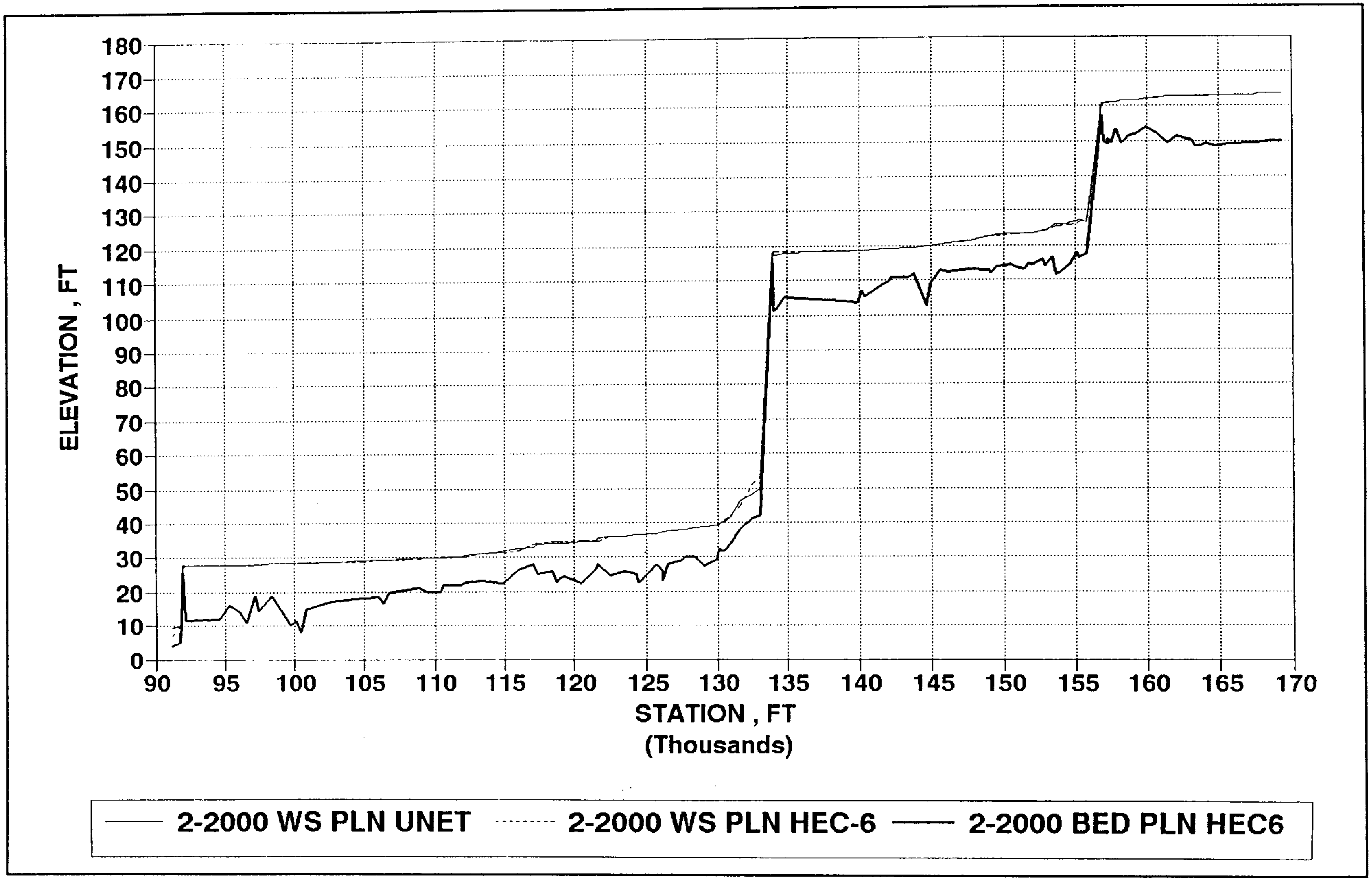


Figure B1. Plan condition verification, bed and water-surface profiles, Branch 1, Passaic River

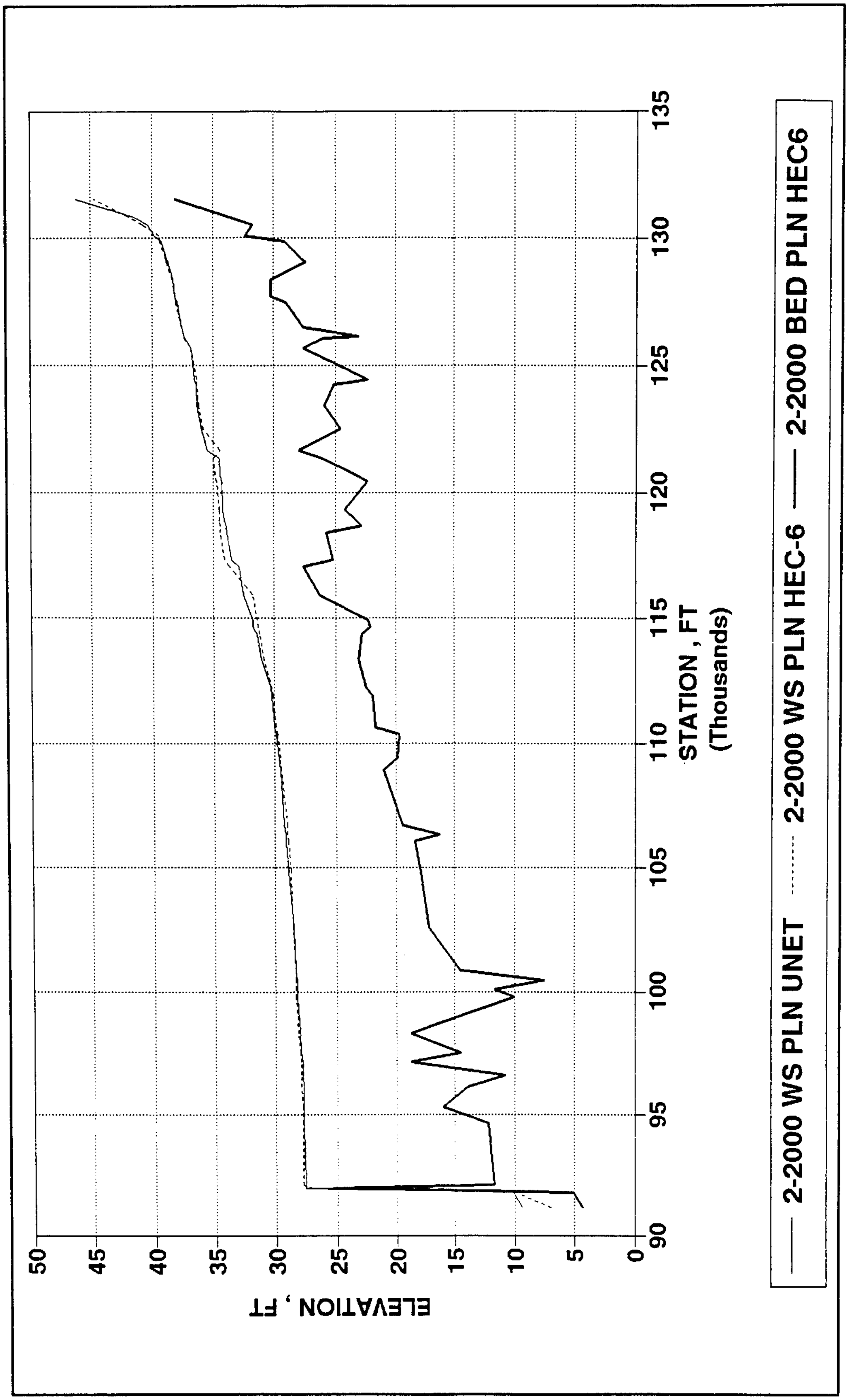


Figure B2. Plan condition verification, bed and water-surface profiles, Branch 1, Passaic River, Dundee Dam

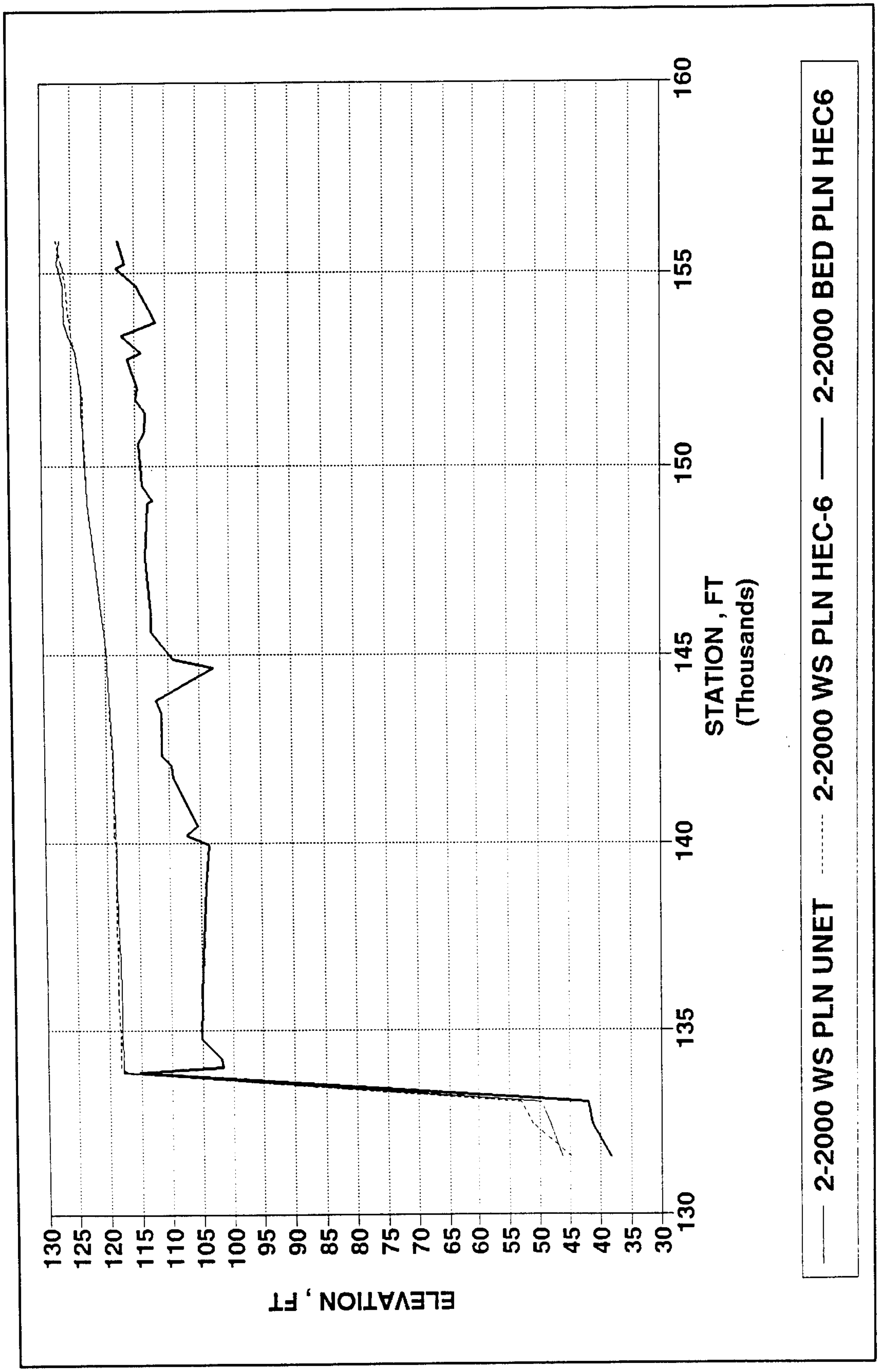


Figure B3. Plan condition verification, bed and water-surface profiles, Branch 1, Passaic River, S.U.M. Dam

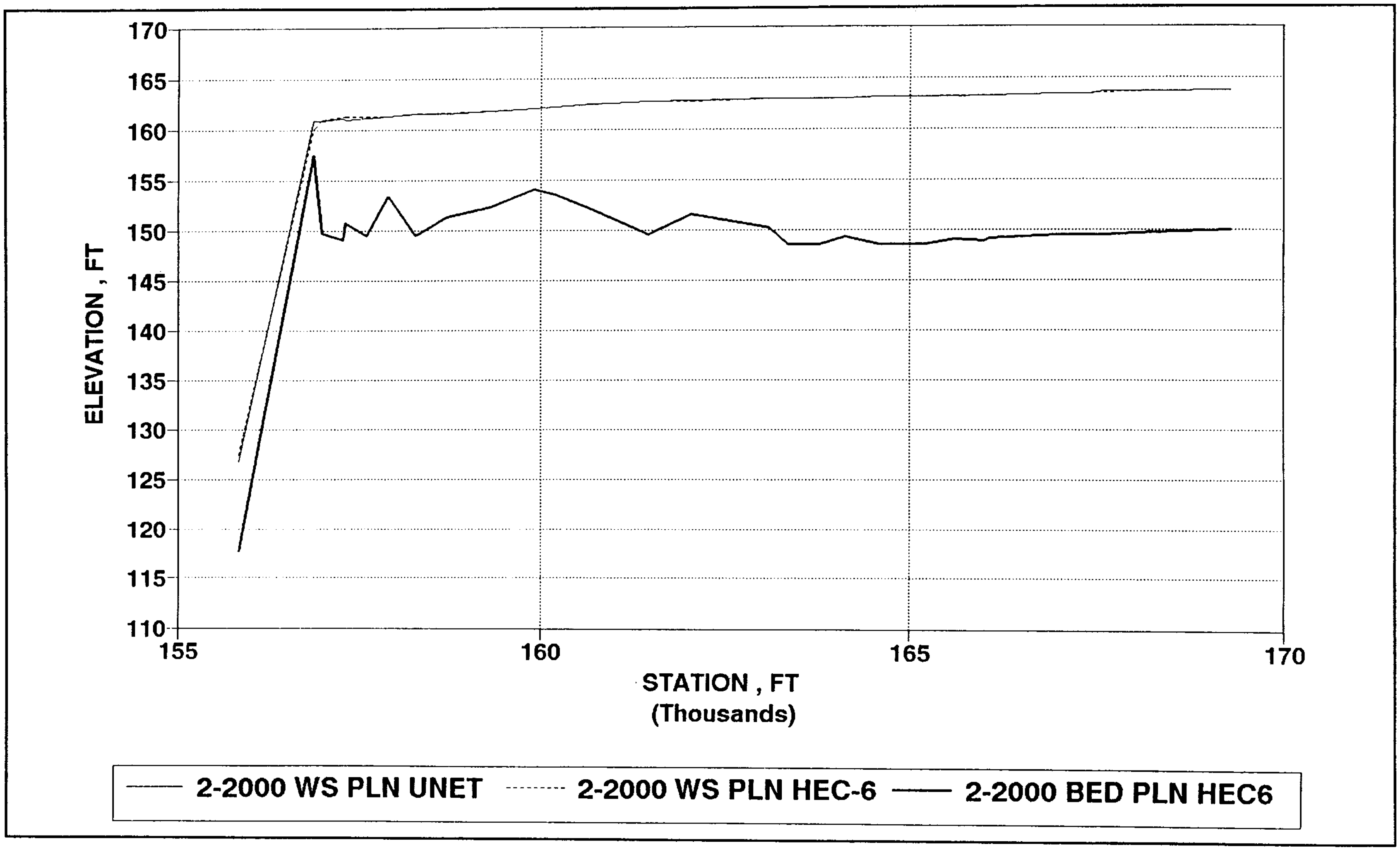


Figure B4. Plan condition verification, bed and water-surface profiles, Branch 1, Passaic River, Beatties Dam

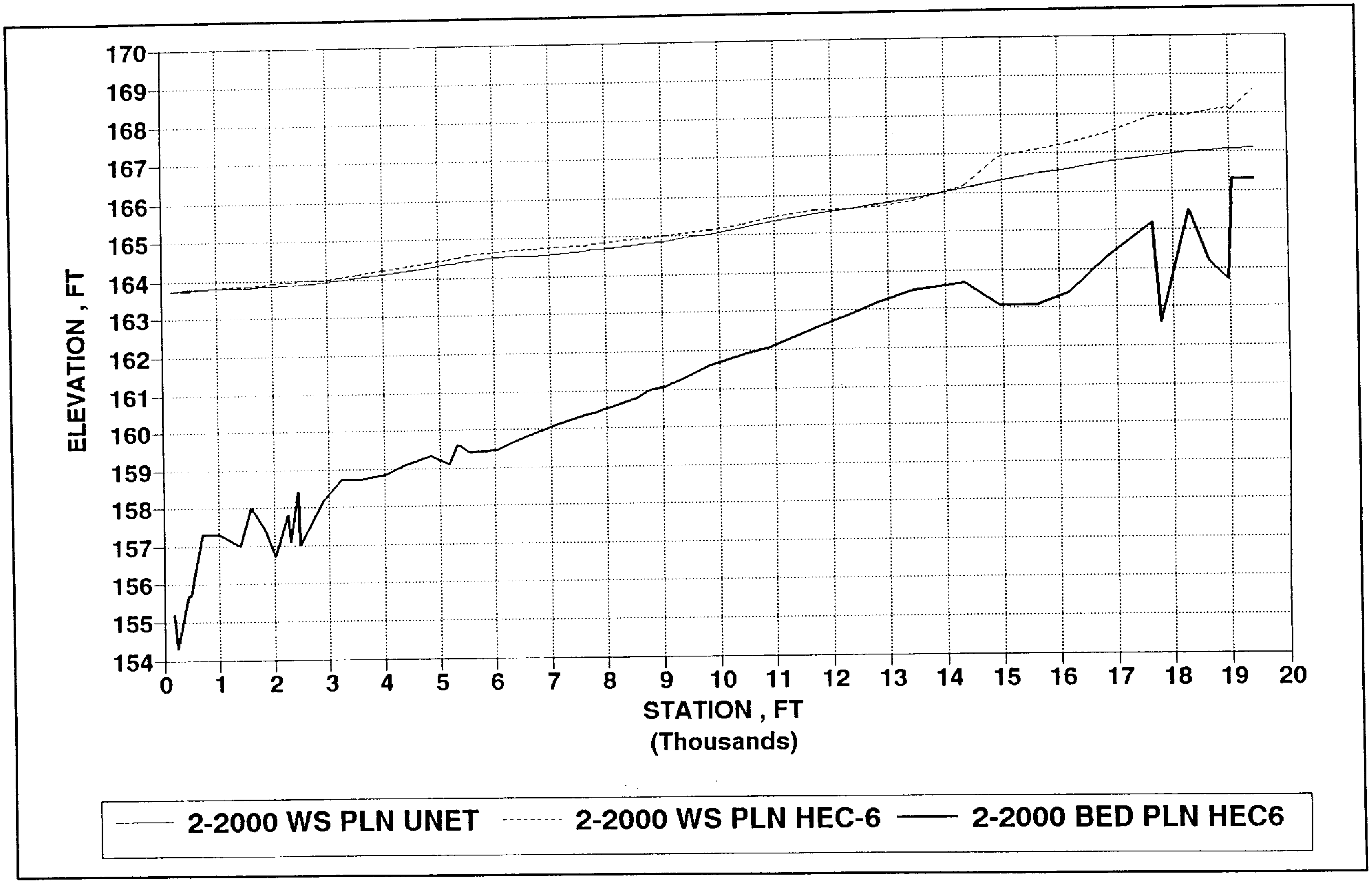


Figure B5. Plan condition verification, bed and water-surface profiles, Branch 2, Deepavaal Brook

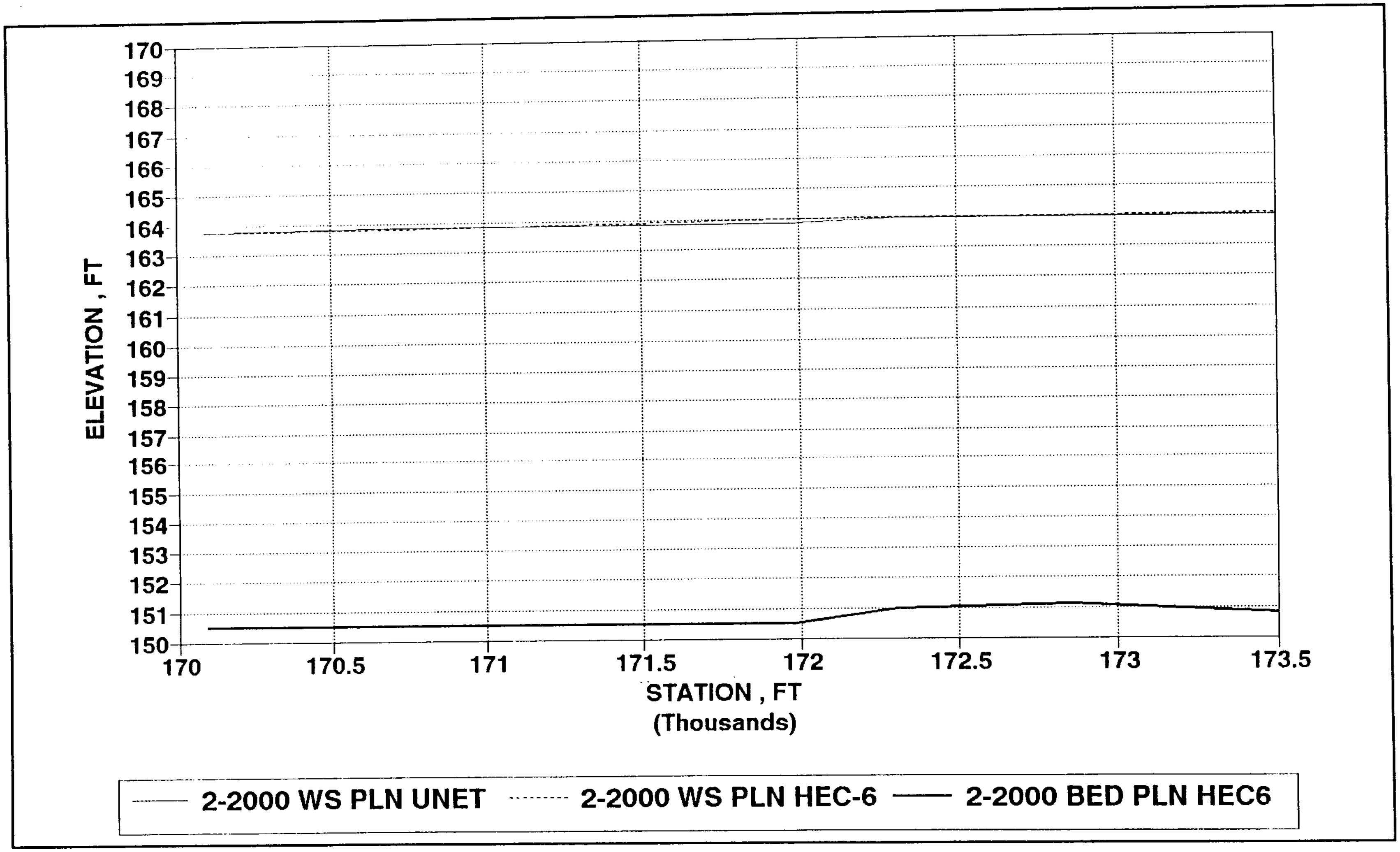


Figure B6. Plan condition verification, bed and water-surface profiles, Branch 3, Passaic River

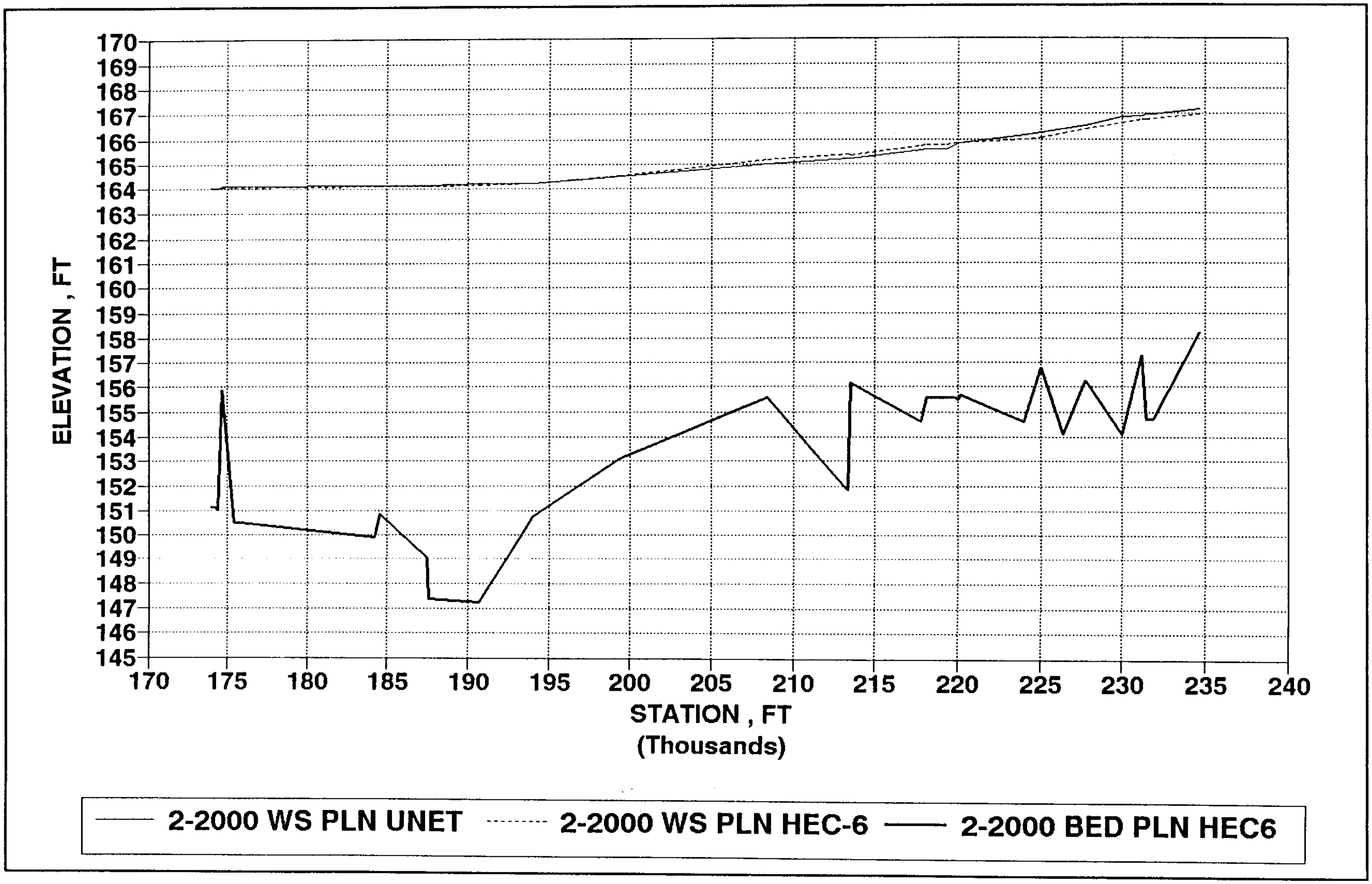


Figure B7. Plan condition verification, bed and water-surface profiles, Branch 4, Passaic River

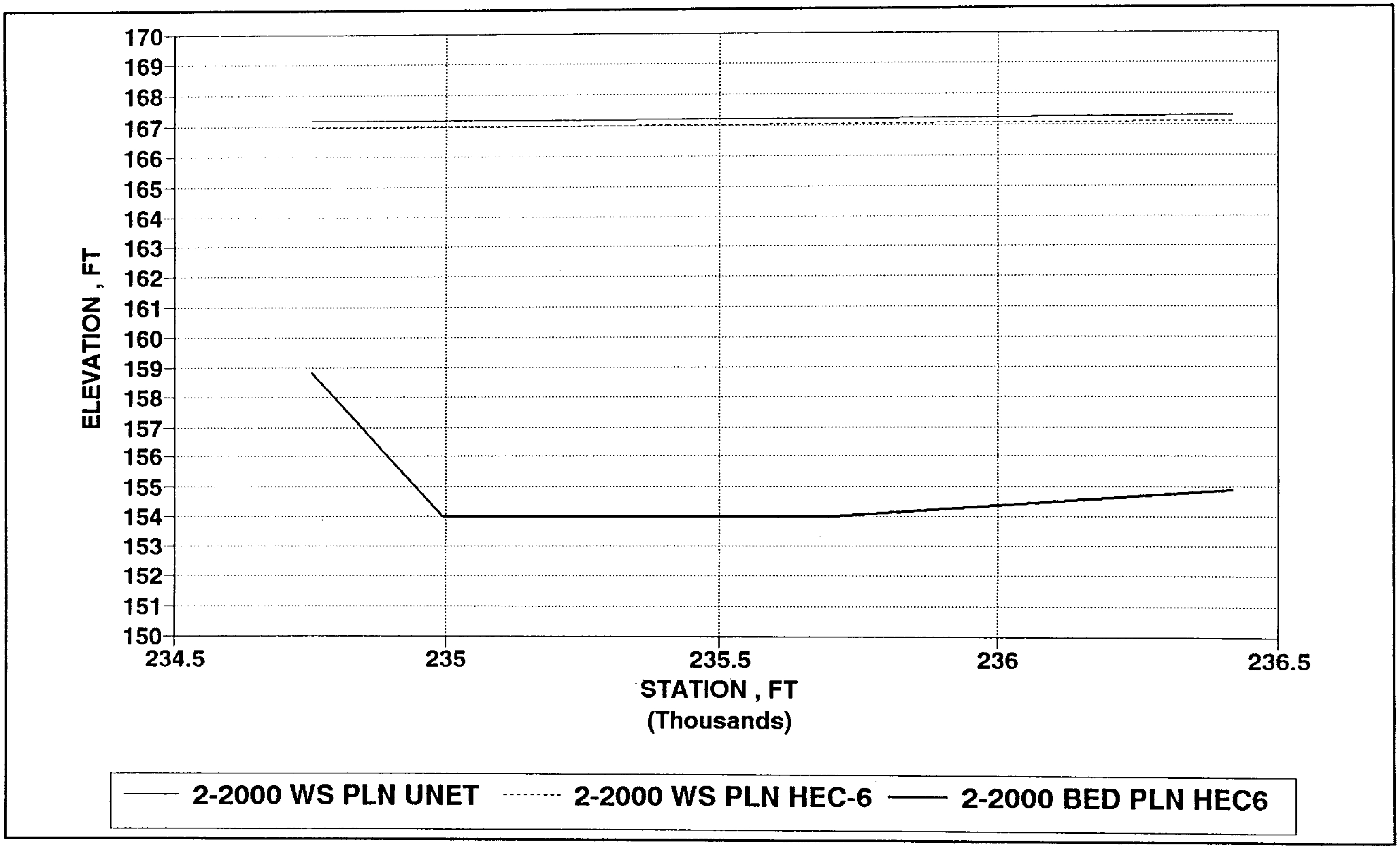


Figure B8. Plan condition verification, bed and water-surface profiles, Branch 5, Passaic River

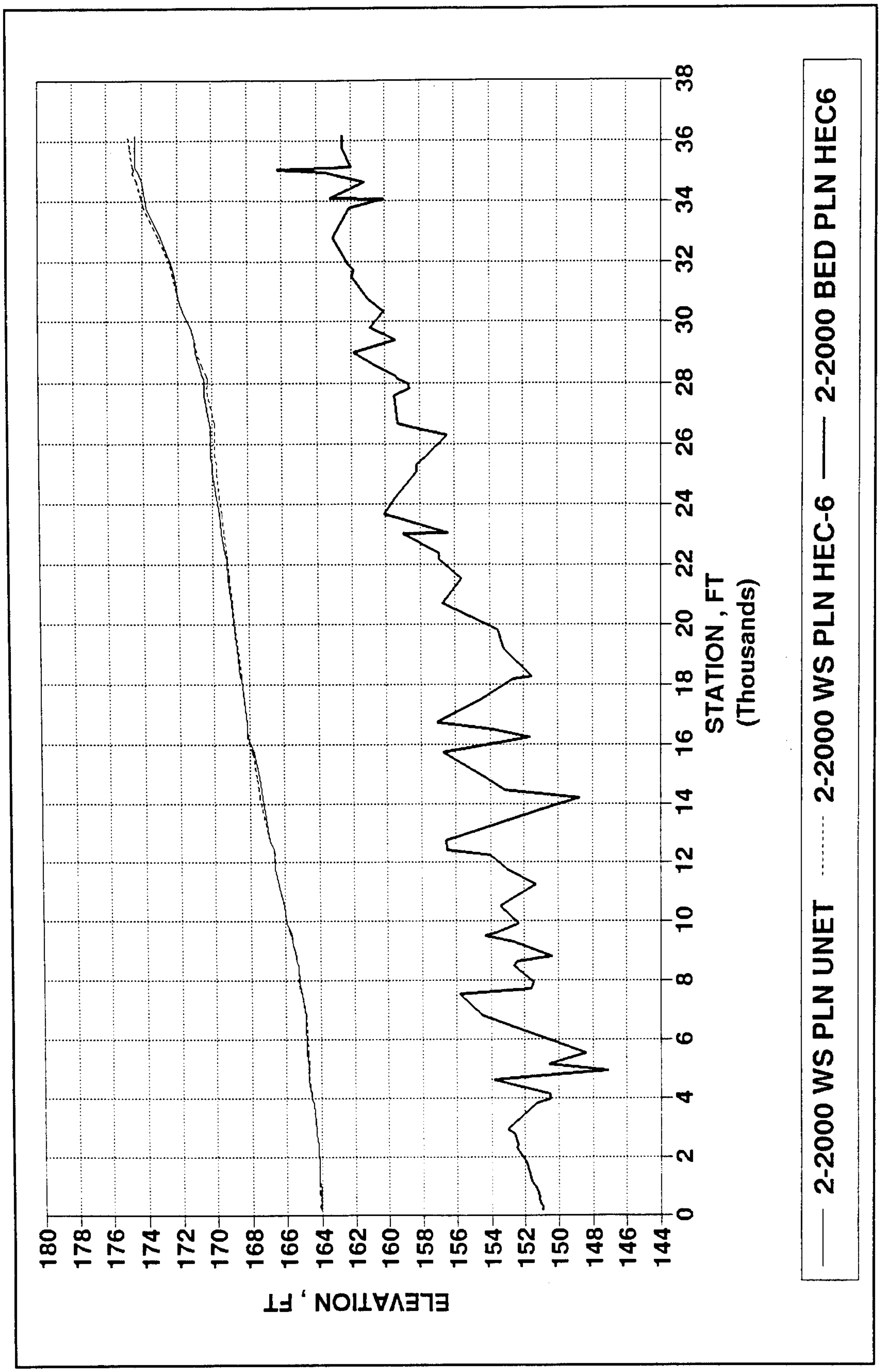


Figure B9. Plan condition verification, bed and water-surface profiles, Branch 6, Pompton River

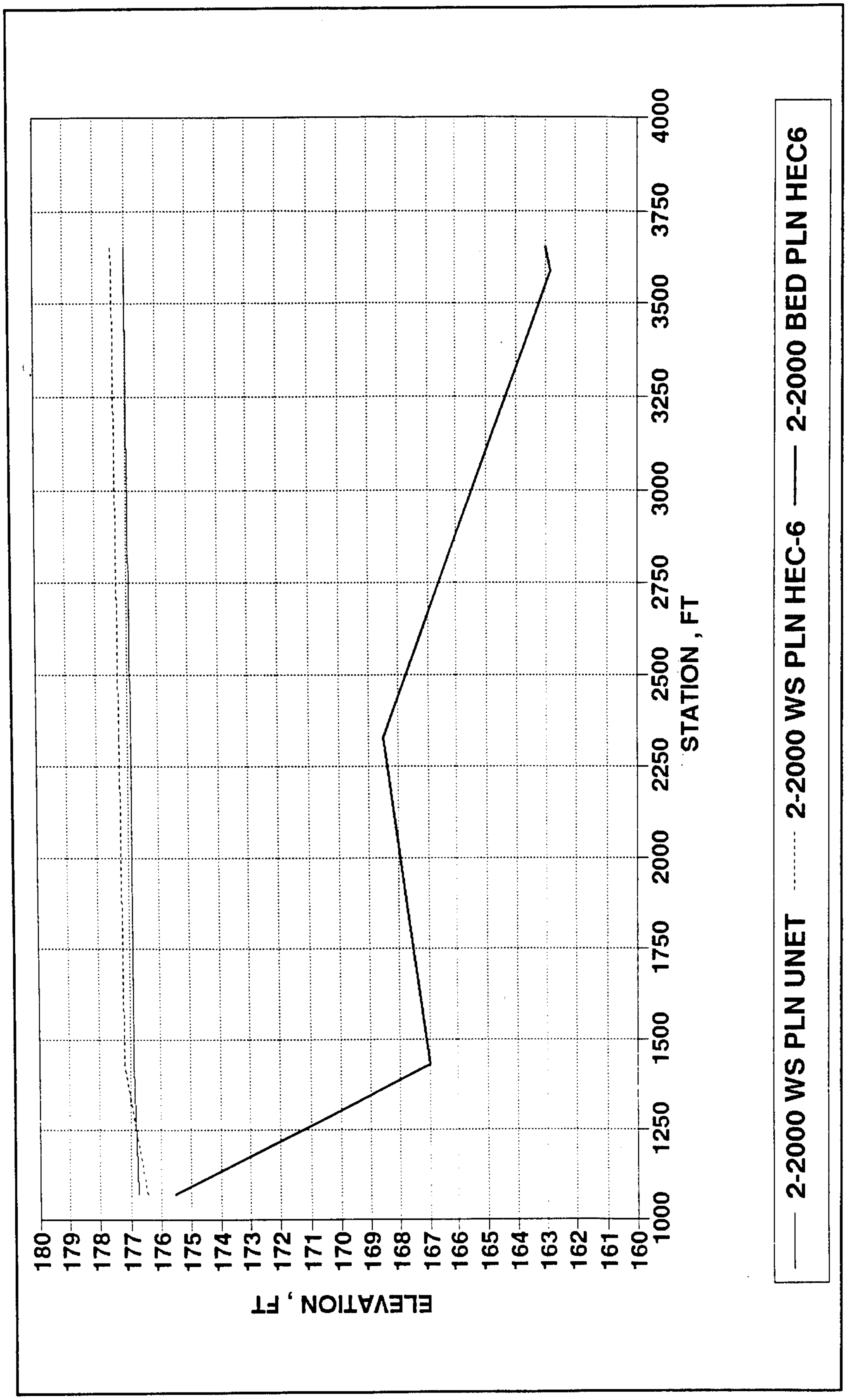


Figure B10. Plan condition verification, bed and water-surface profiles, Branch 7, Ramapo River

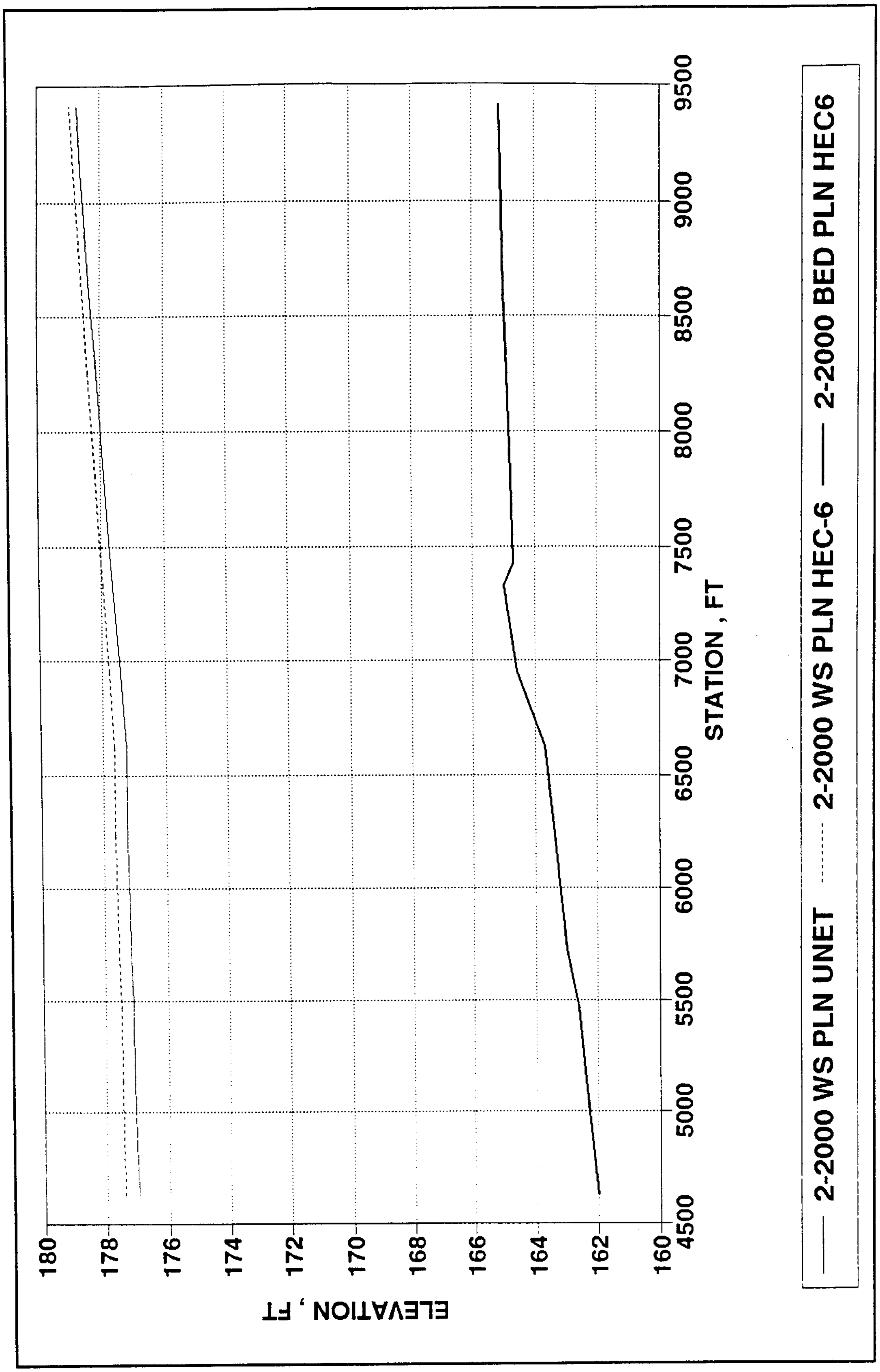


Figure B11. Plan condition verification, bed and water-surface profiles, Branch 8, Ramapo River

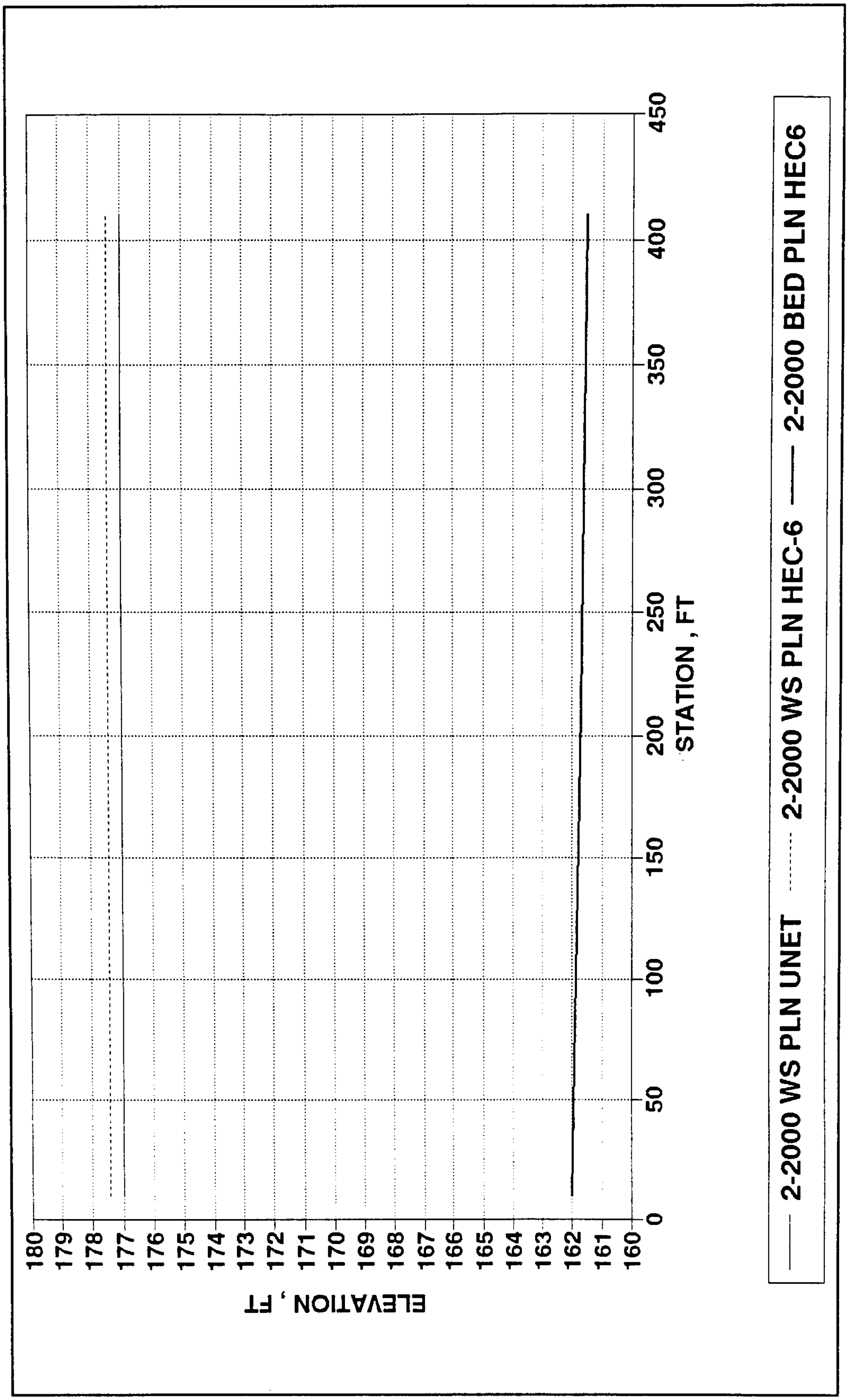


Figure B12. Plan condition verification, bed and water-surface profiles, Branch 9

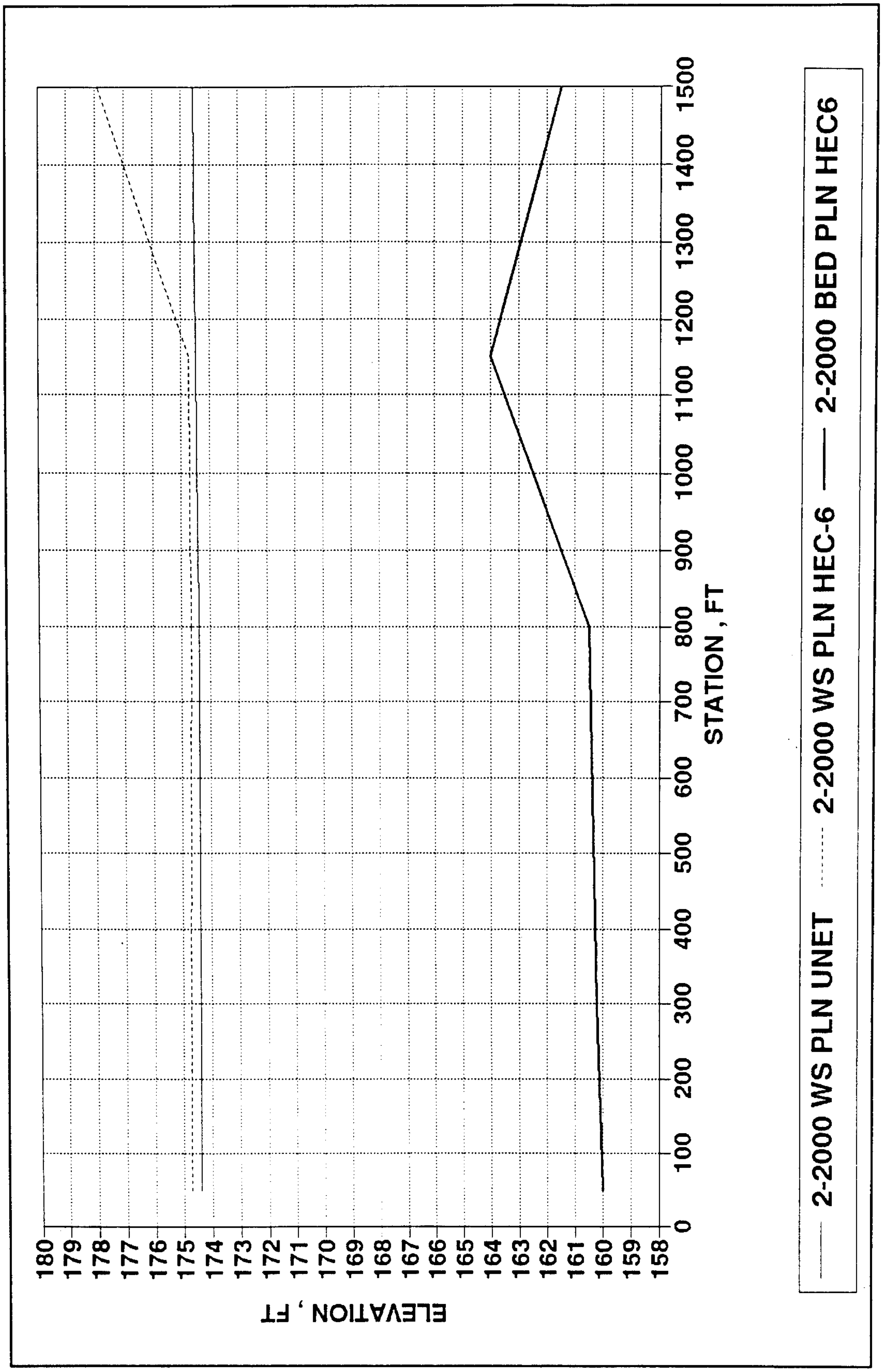


Figure B13. Plan condition verification, bed and water-surface profiles, Branch 10, Bypass

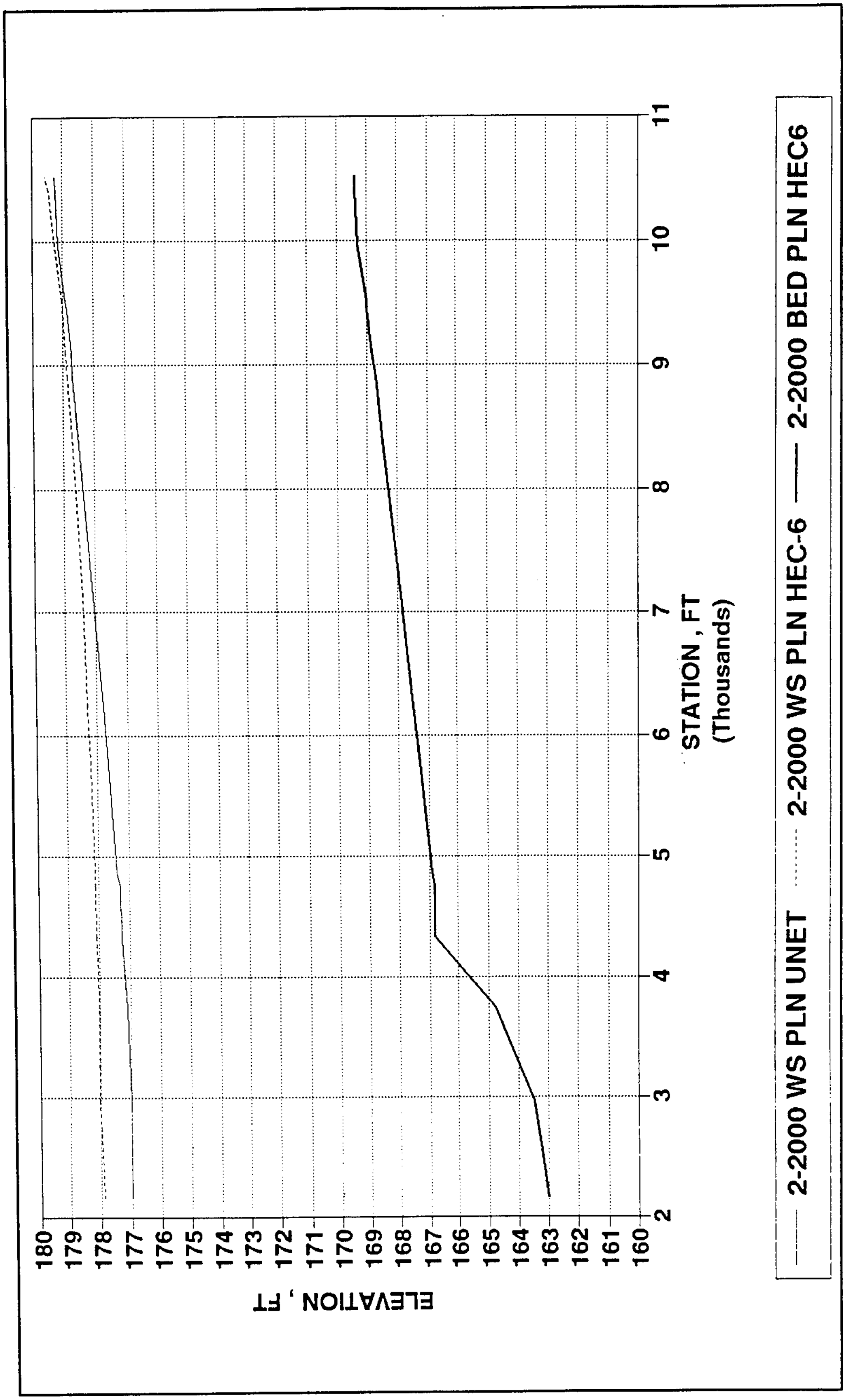


Figure B14. Plan condition verification, bed and water-surface profiles, Branch 11, Pequannock River

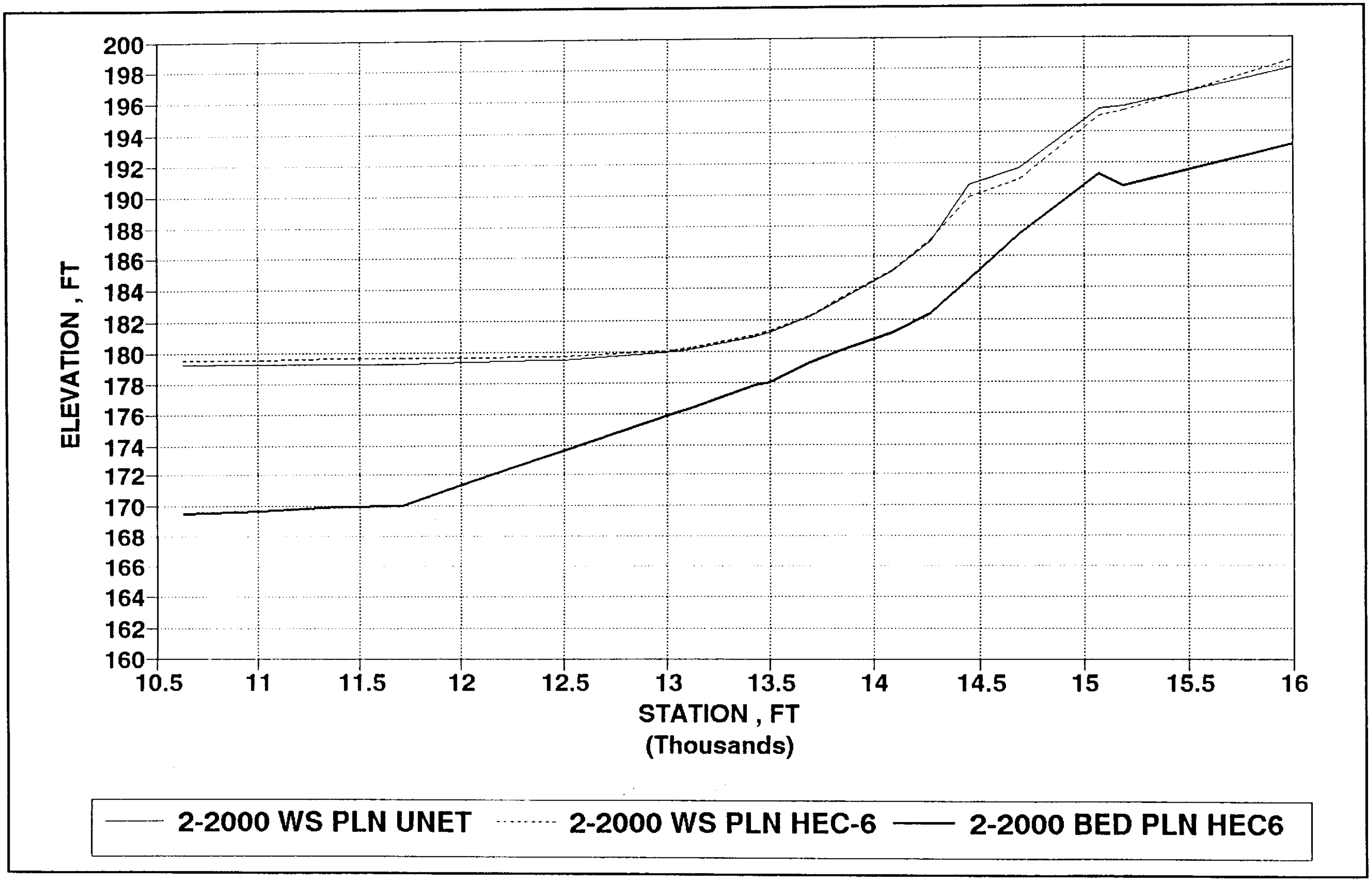


Figure B15. Plan condition verification, bed and water-surface profiles, Branch 12, Pequannock River

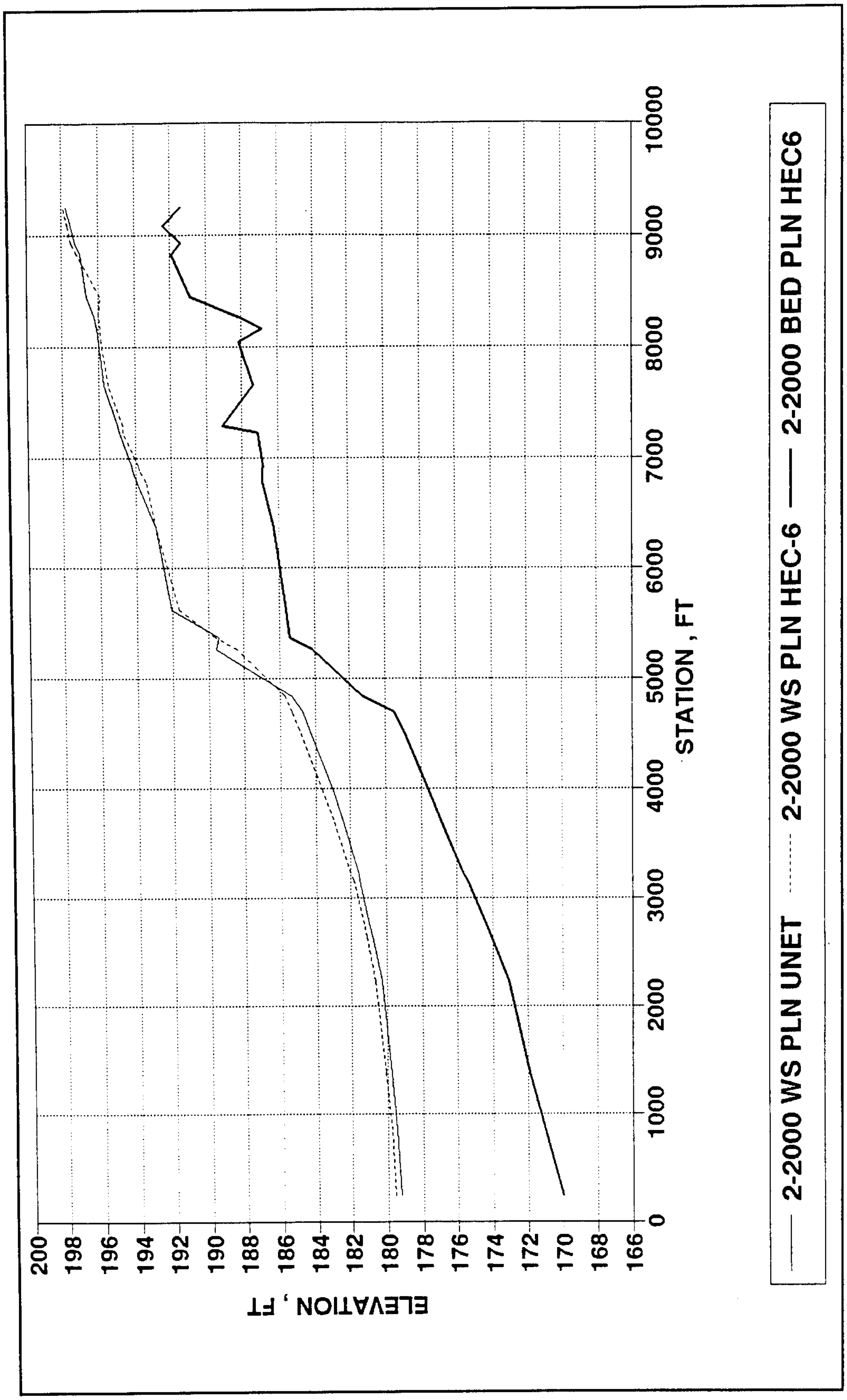


Figure B16. Plan condition verification, bed and water-surface profiles, Branch 13, Wanaque River

Appendix C
Base Long-Term Simulation

This appendix contains graphs for river branches 1 to 13 showing the differences between base beds and base water surfaces from the initial condition (year 2000) to the year 2050 using a 2-year steady state peak flow. To convert feet to meters, multiply by 0.3048.

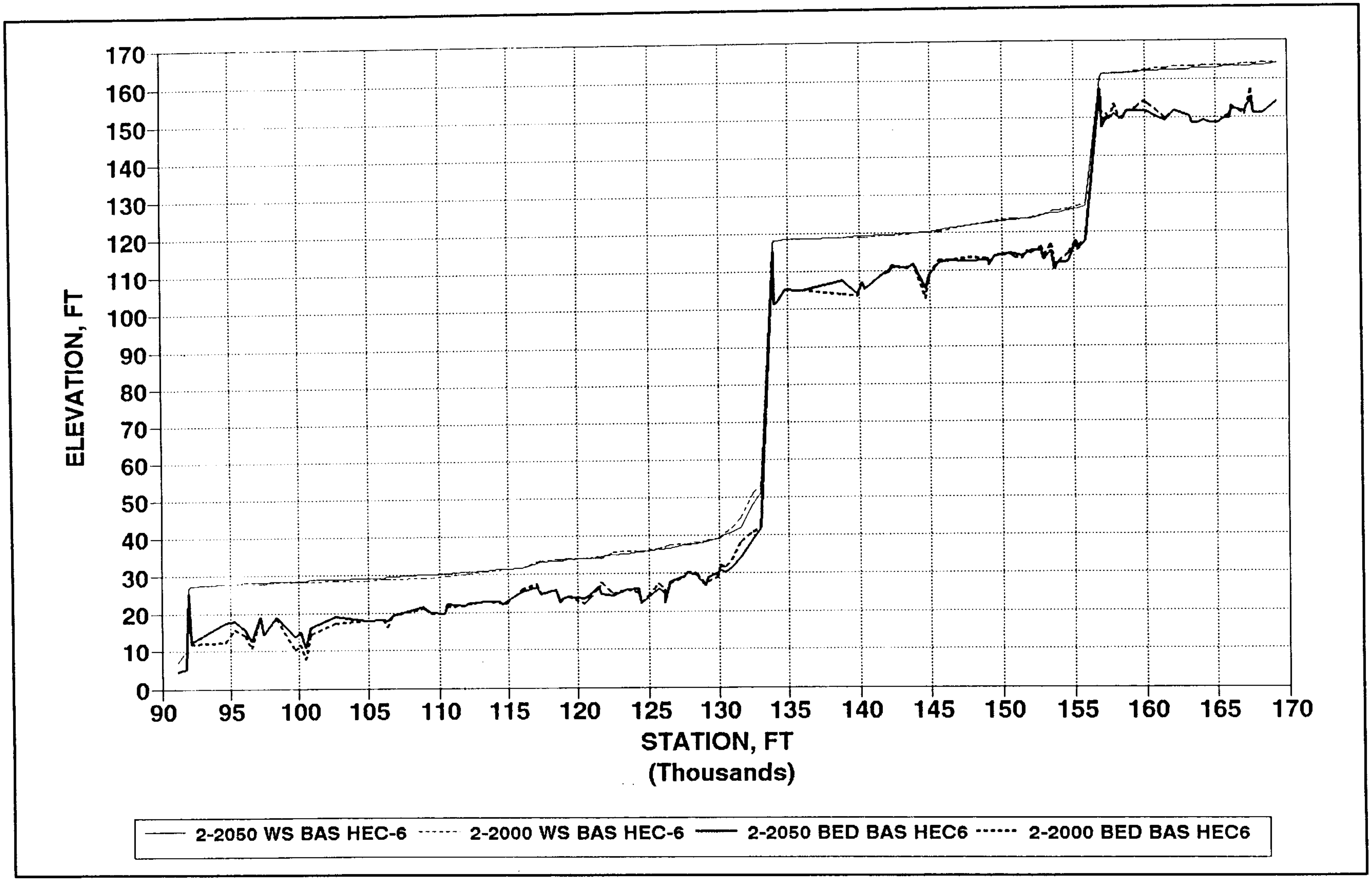


Figure C1. Base long-term simulation, bed and water-surface profiles, Branch 1, Passaic River

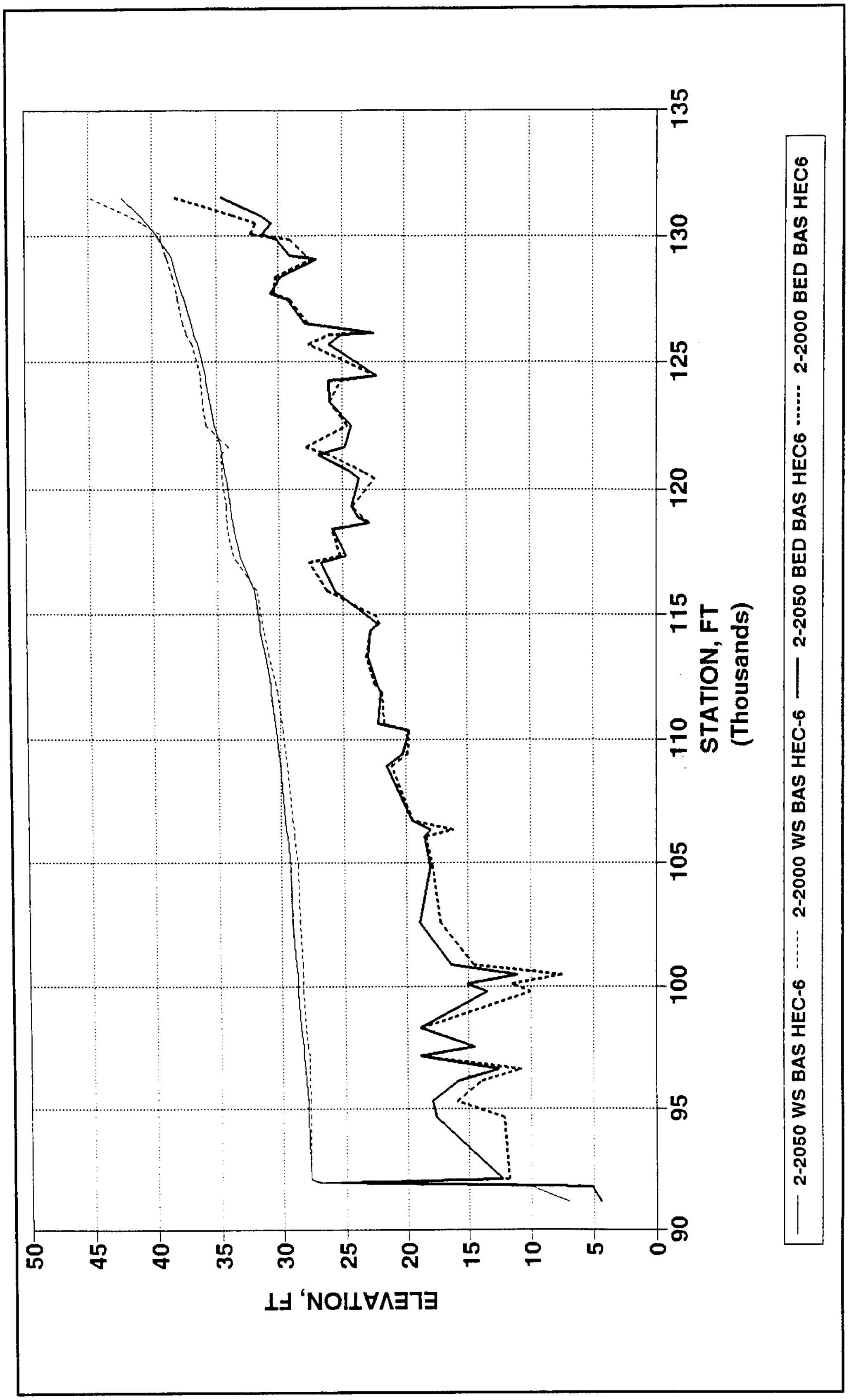


Figure C2. Base long-term simulation, bed and water-surface profiles, Branch 1, Passaic River, Dundee Dam

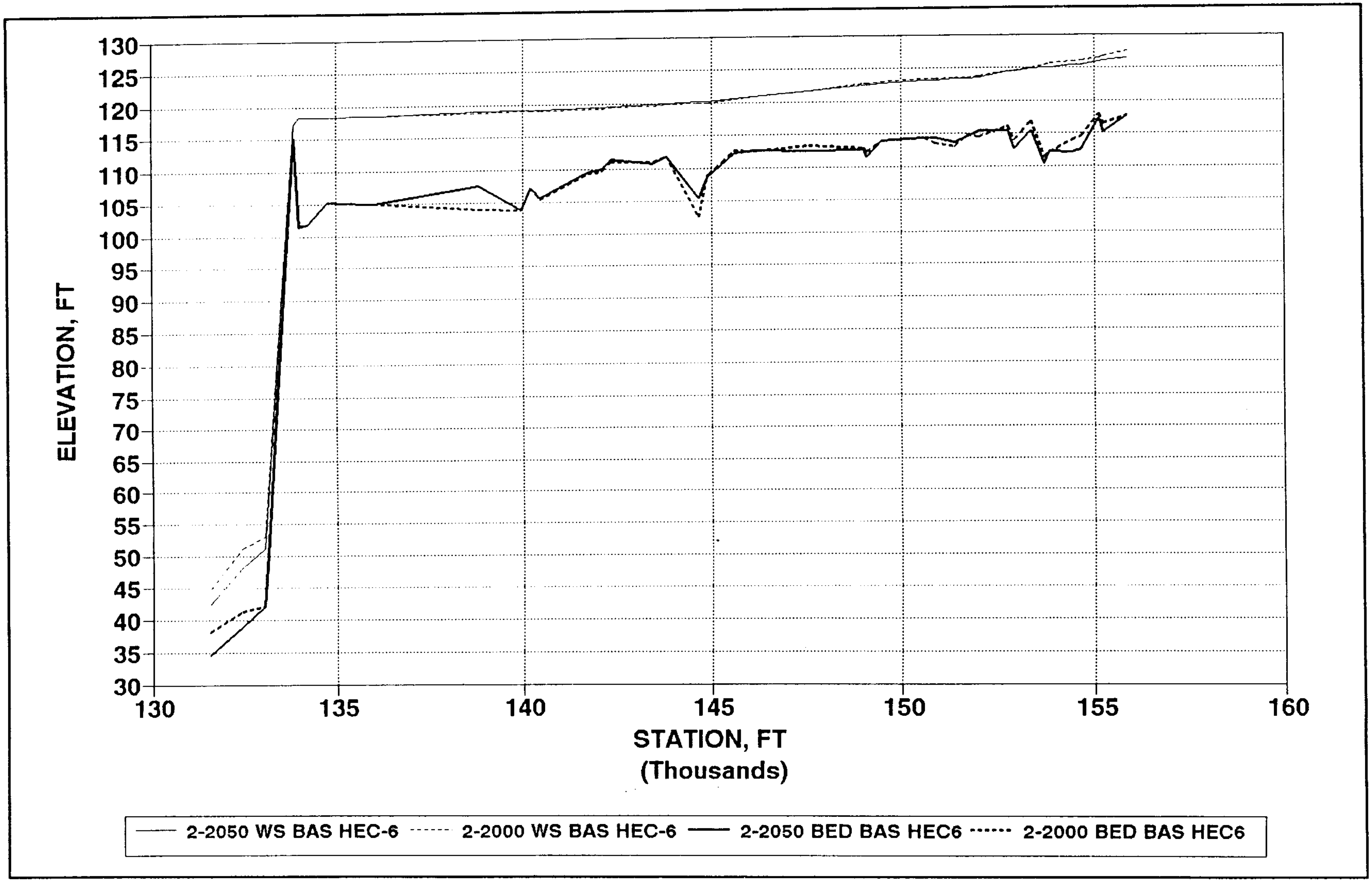


Figure C3. Base long-term simulation, bed and water-surface profiles, Branch 1, Passaic River, S.U.M. Dam

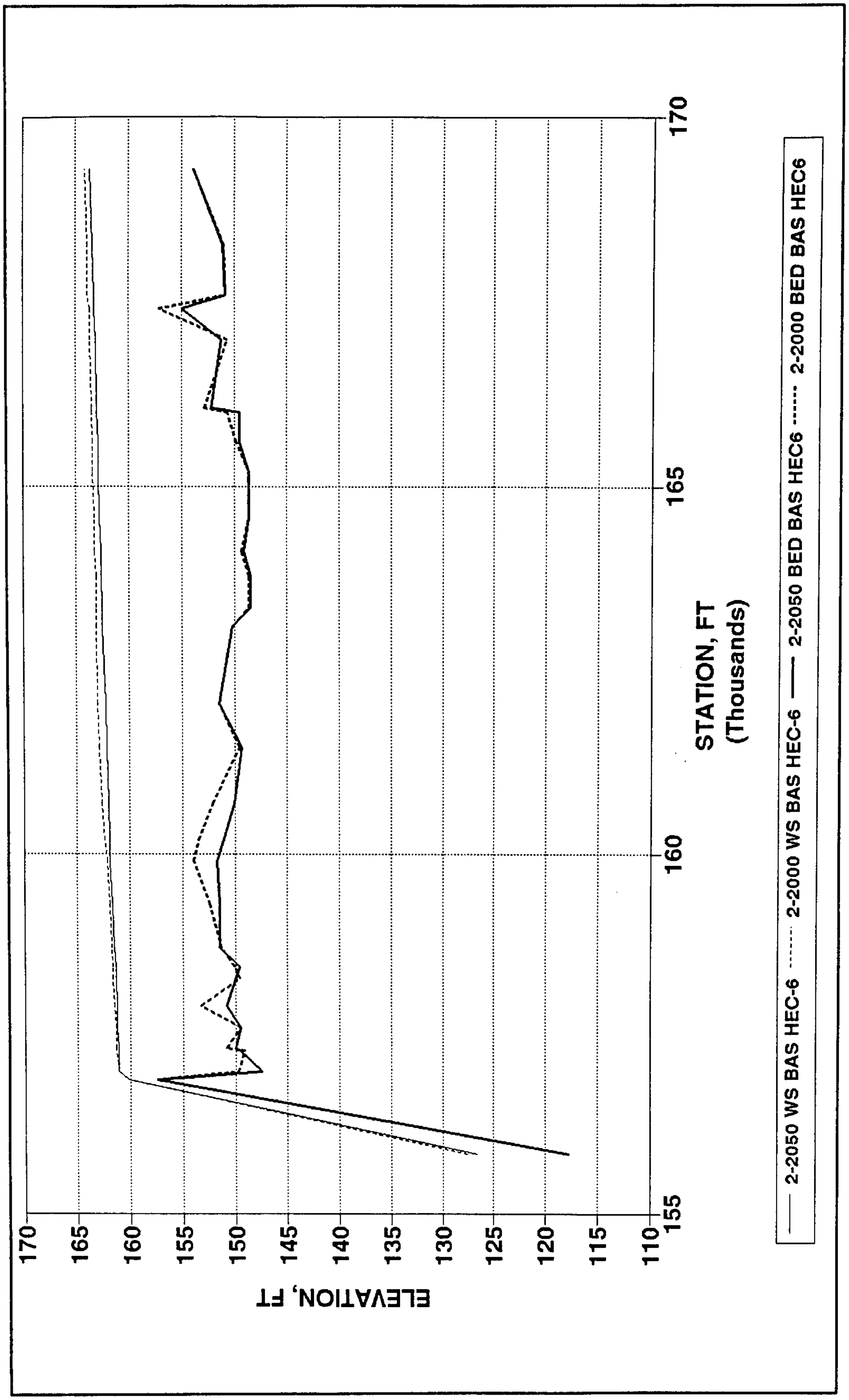


Figure C4. Base long-term simulation, bed and water-surface profiles, Branch 1, Passaic River, Beatties Dam

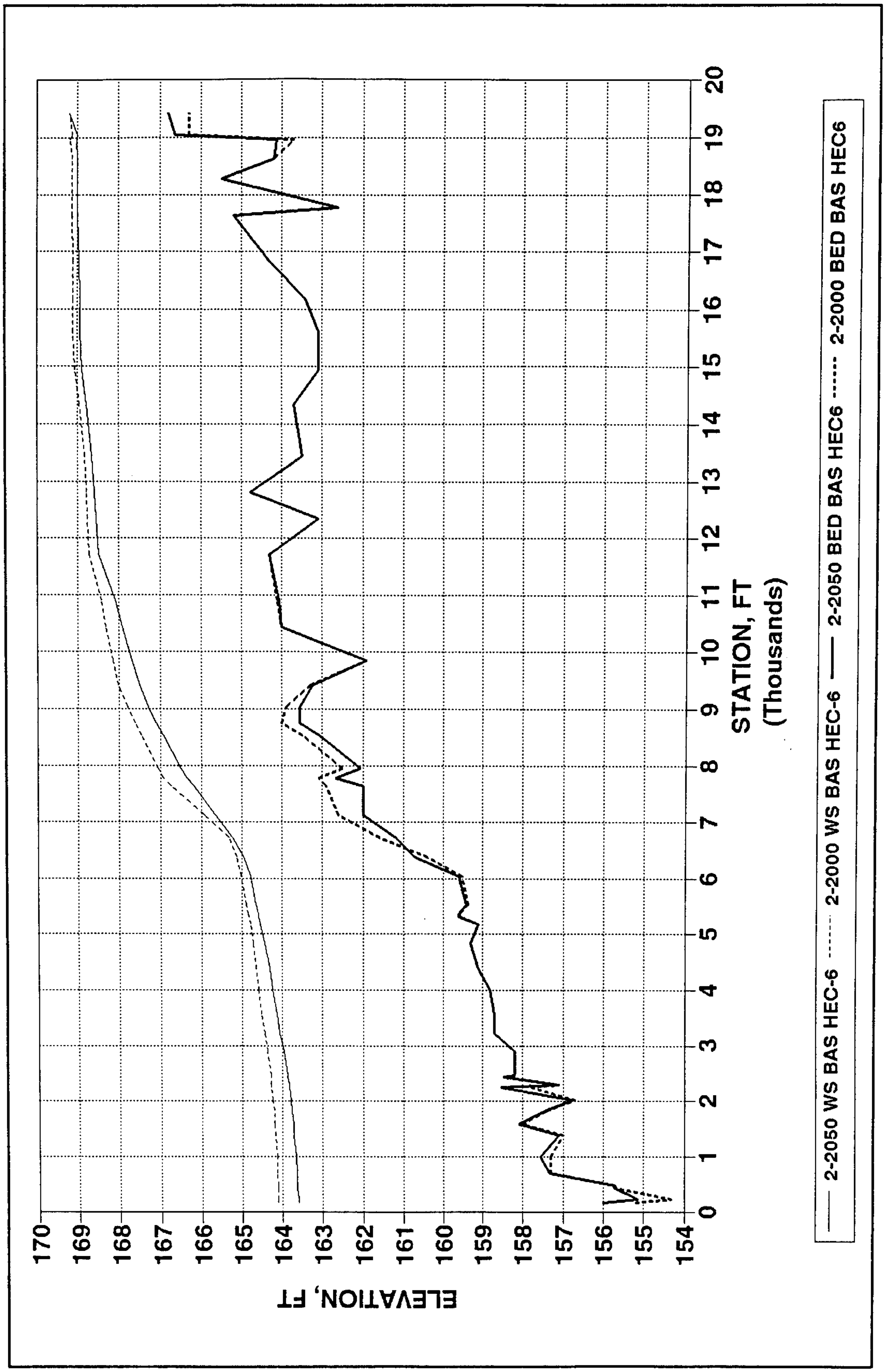


Figure C5. Base long-term simulation, bed and water-surface profiles, Branch 2, Deepavaal Brook

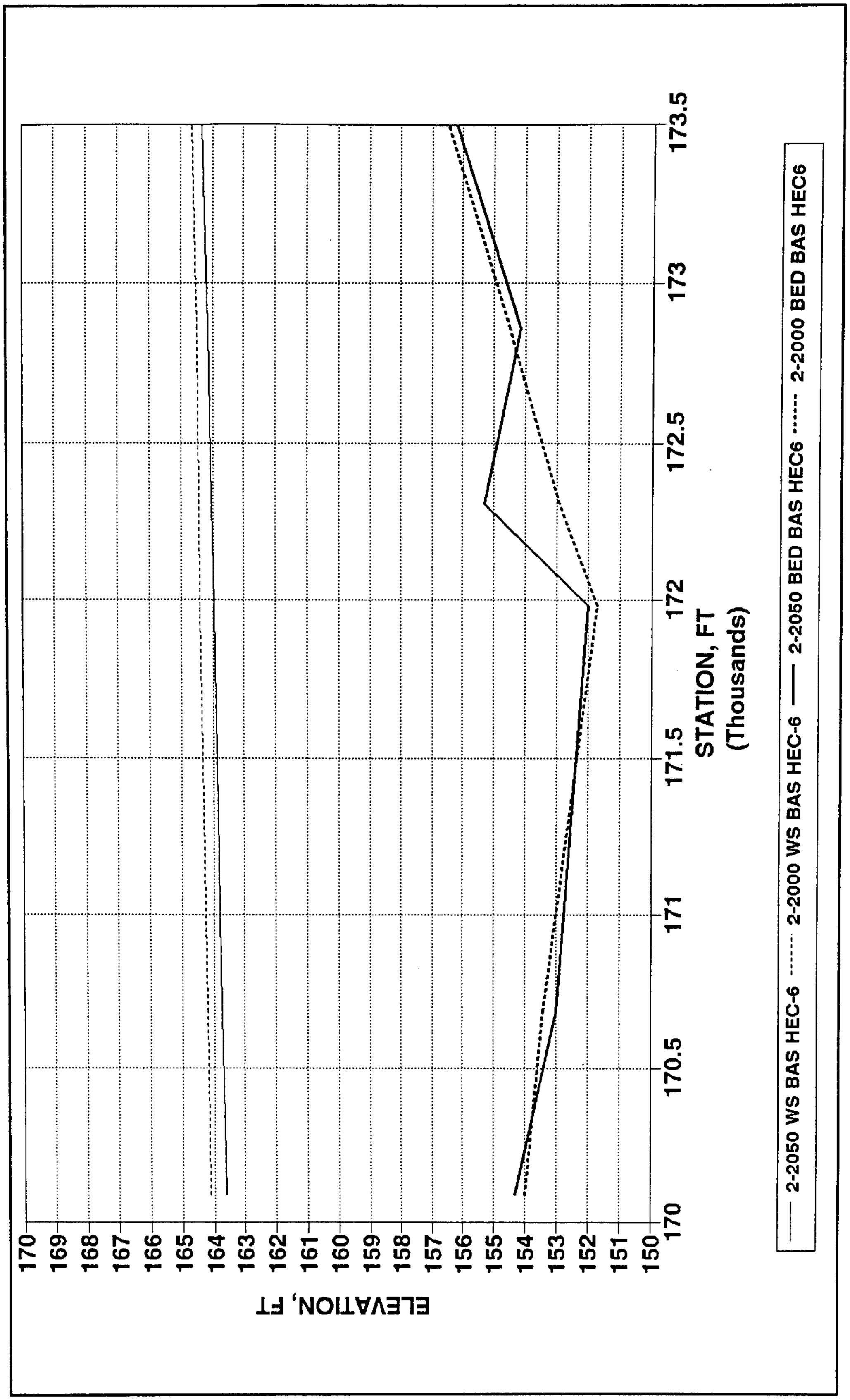


Figure C6. Base long-term simulation, bed and water-surface profiles, Branch 3, Passaic River

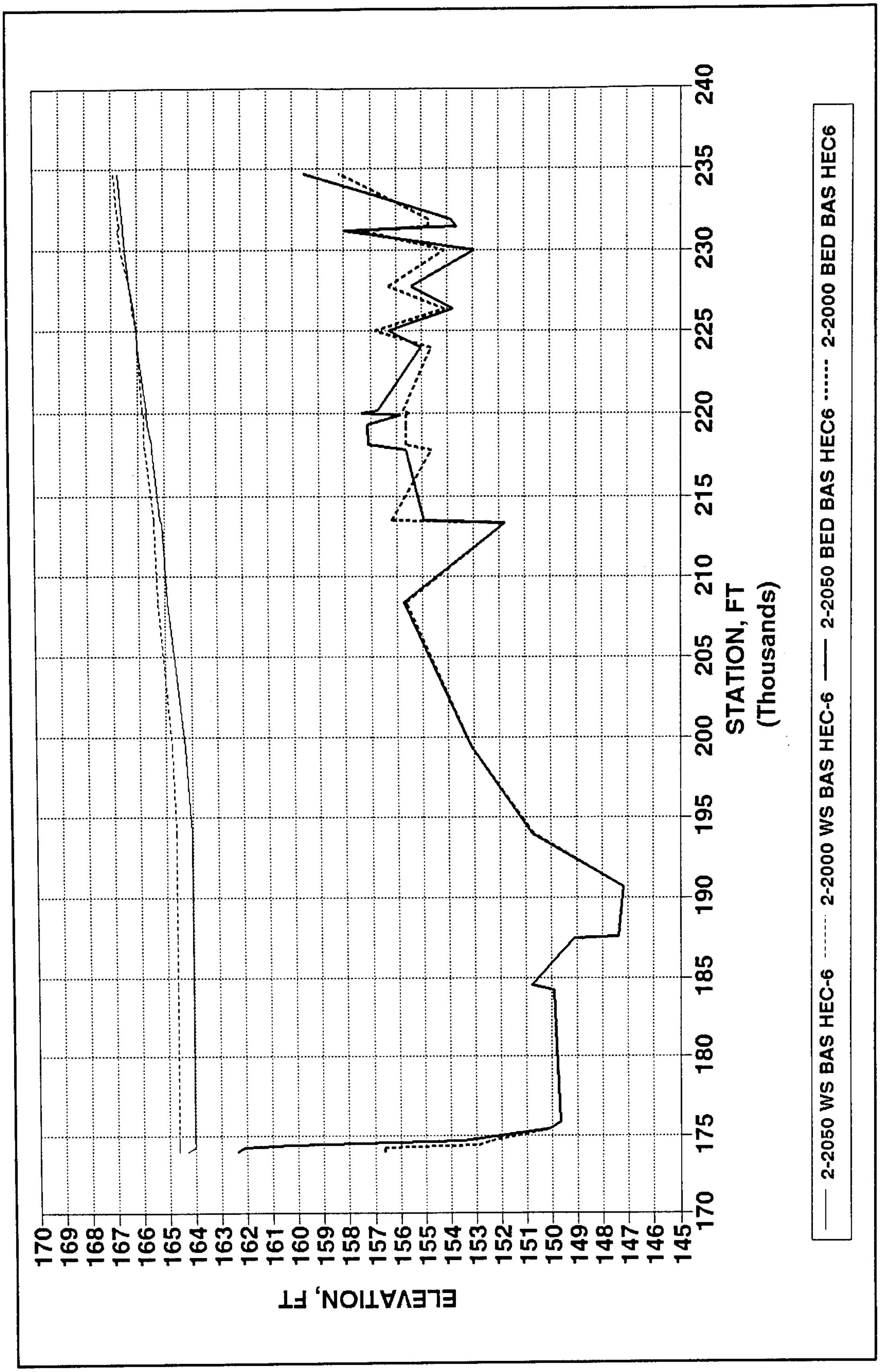


Figure C7. Base long-term simulation, bed and water-surface profiles, Branch 4, Passaic River

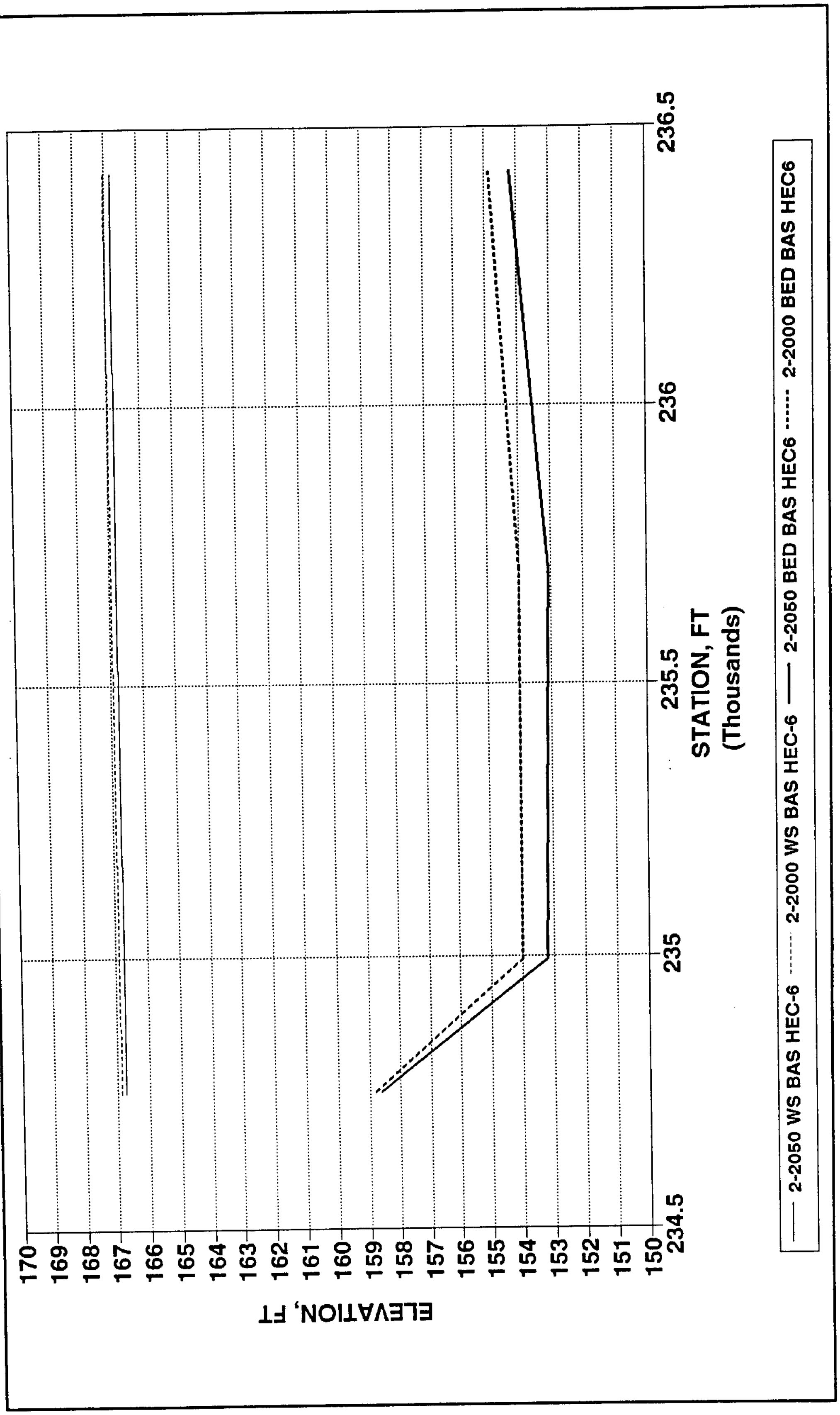


Figure C8. Base long-term simulation, bed and water-surface profiles, Branch 5, Passaic River

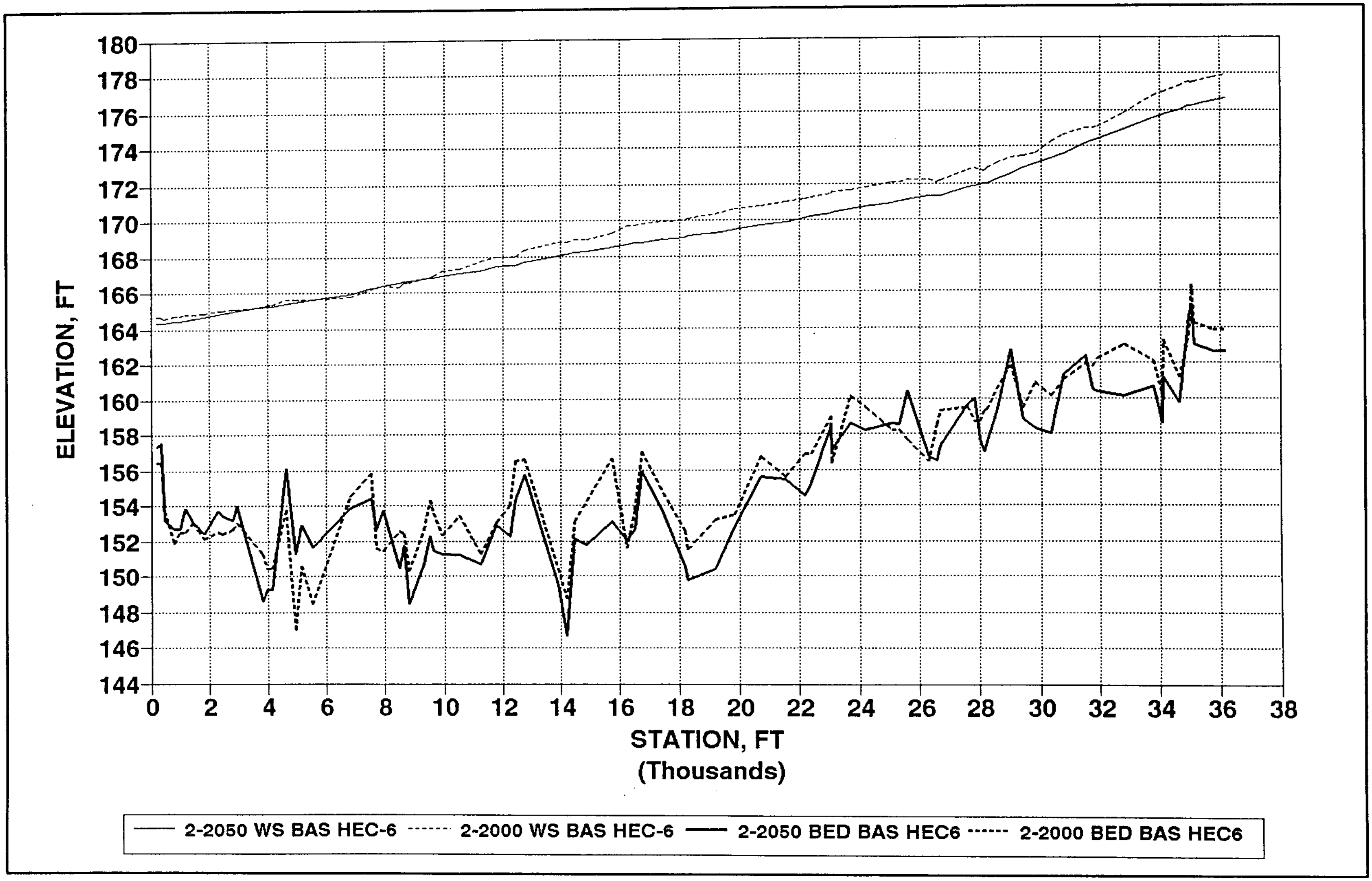


Figure C9. Base long-term simulation, bed and water-surface profiles, Branch 6, Pompton River

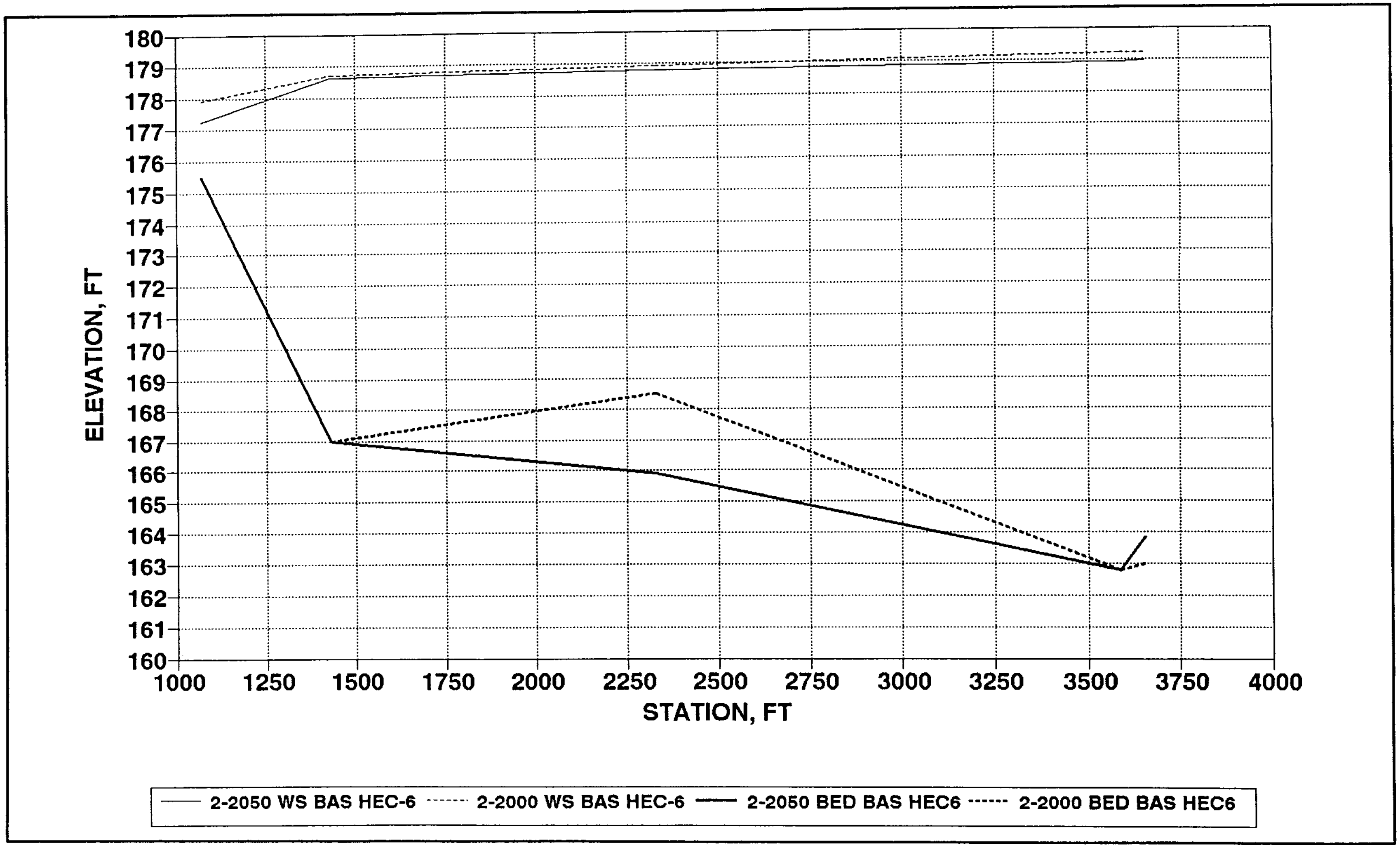


Figure C10. Base long-term simulation, bed and water-surface profiles, Branch 7, Ramapo River

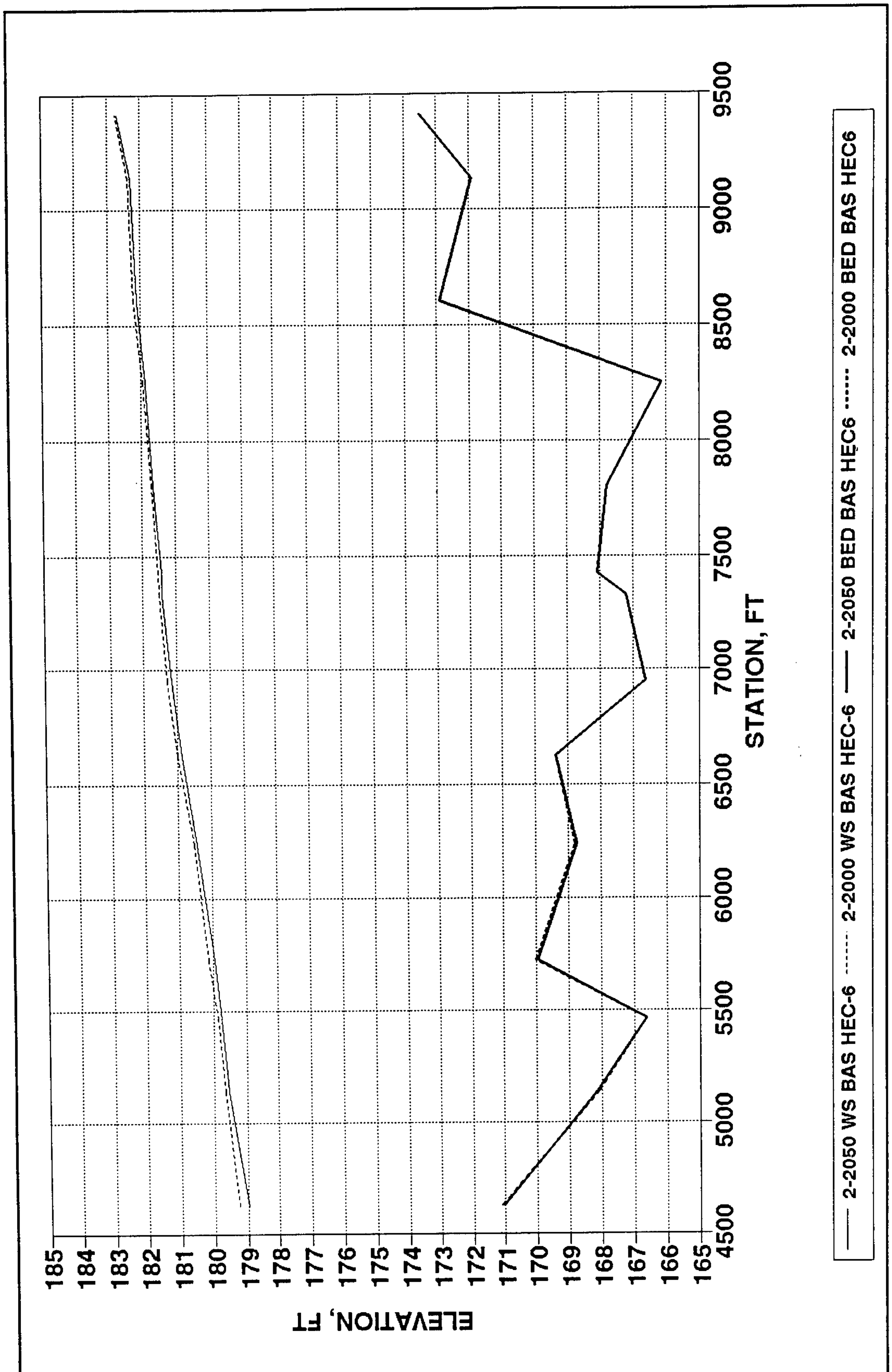


Figure C11. Base long-term simulation, bed and water-surface profiles, Branch 8, Ramapo River

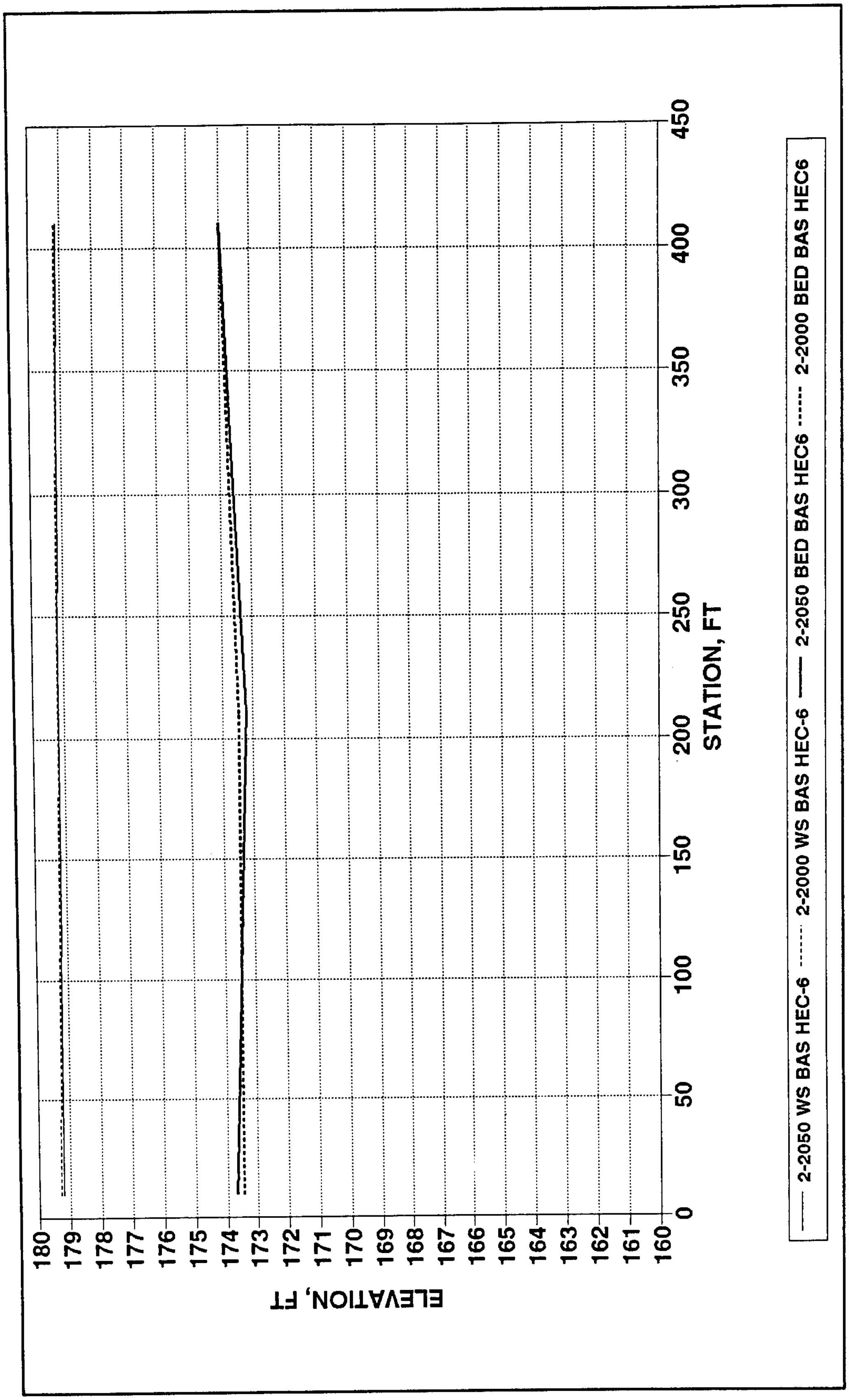


Figure C12. Base long-term simulation, bed and water-surface profiles, Branch 9

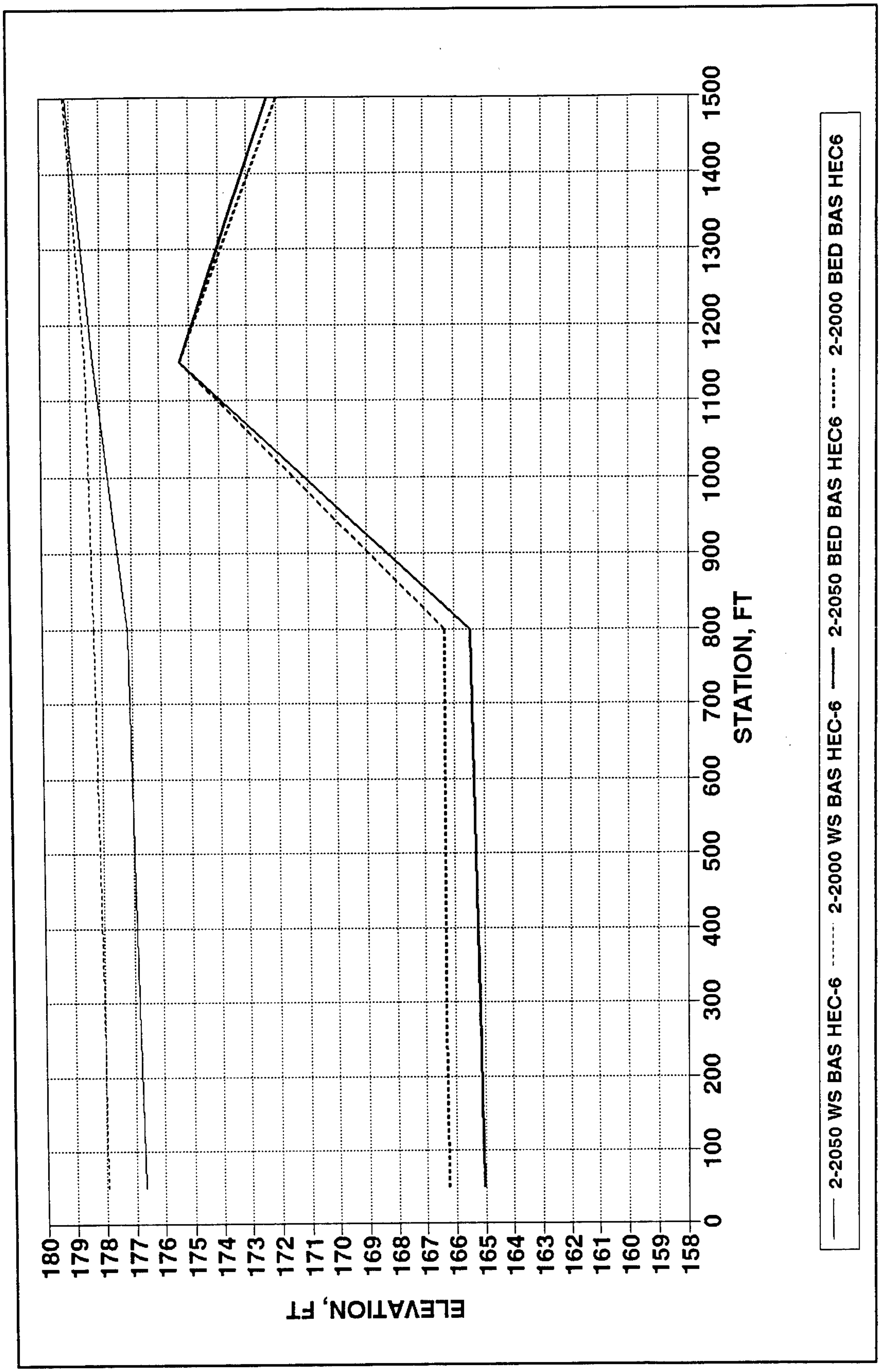


Figure C13. Base long-term simulation, bed and water-surface profiles, Branch 10, Bypass

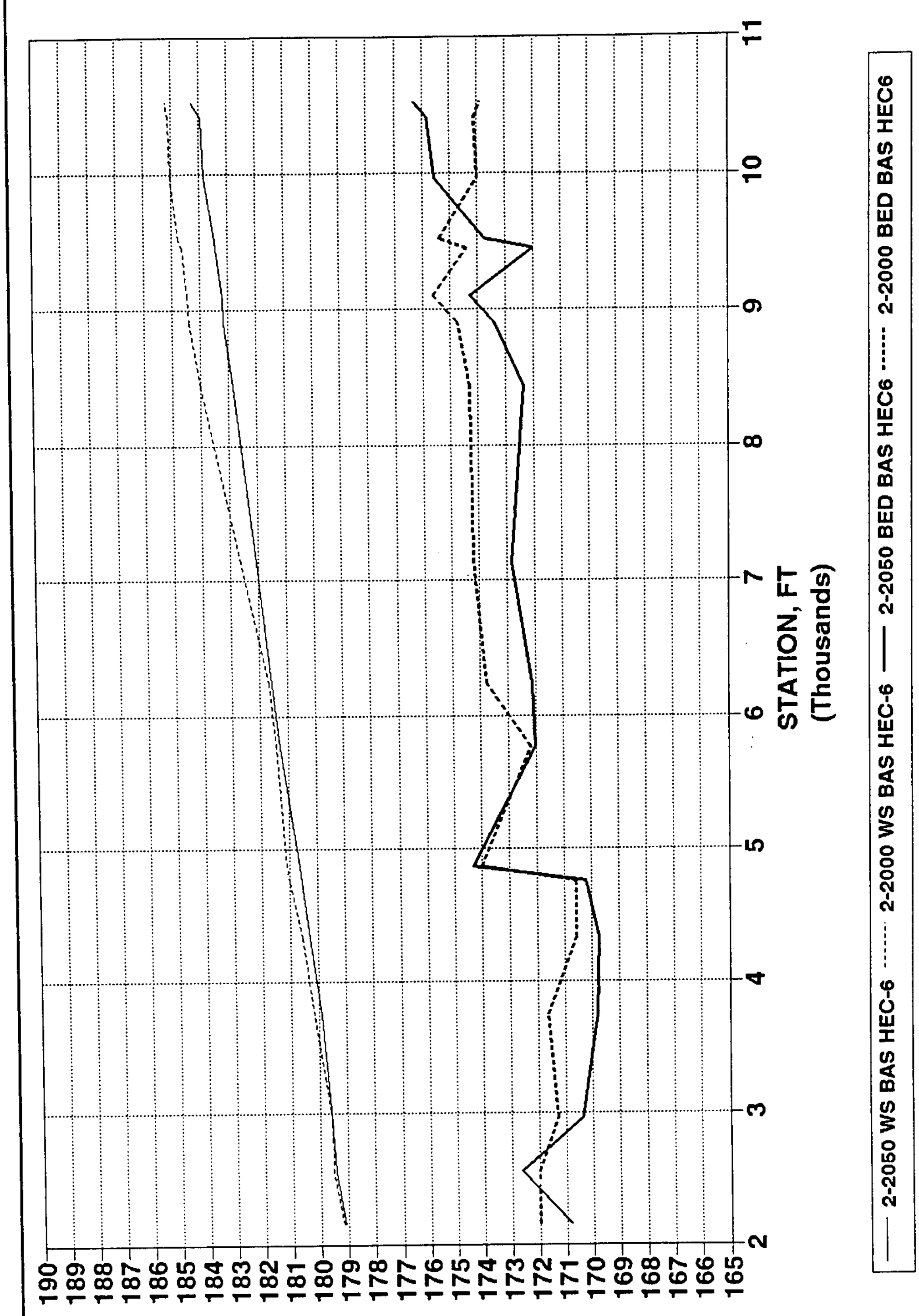


Figure C14. Base long-term simulation, bed and water-surface profiles, Branch 11, Pequannock River

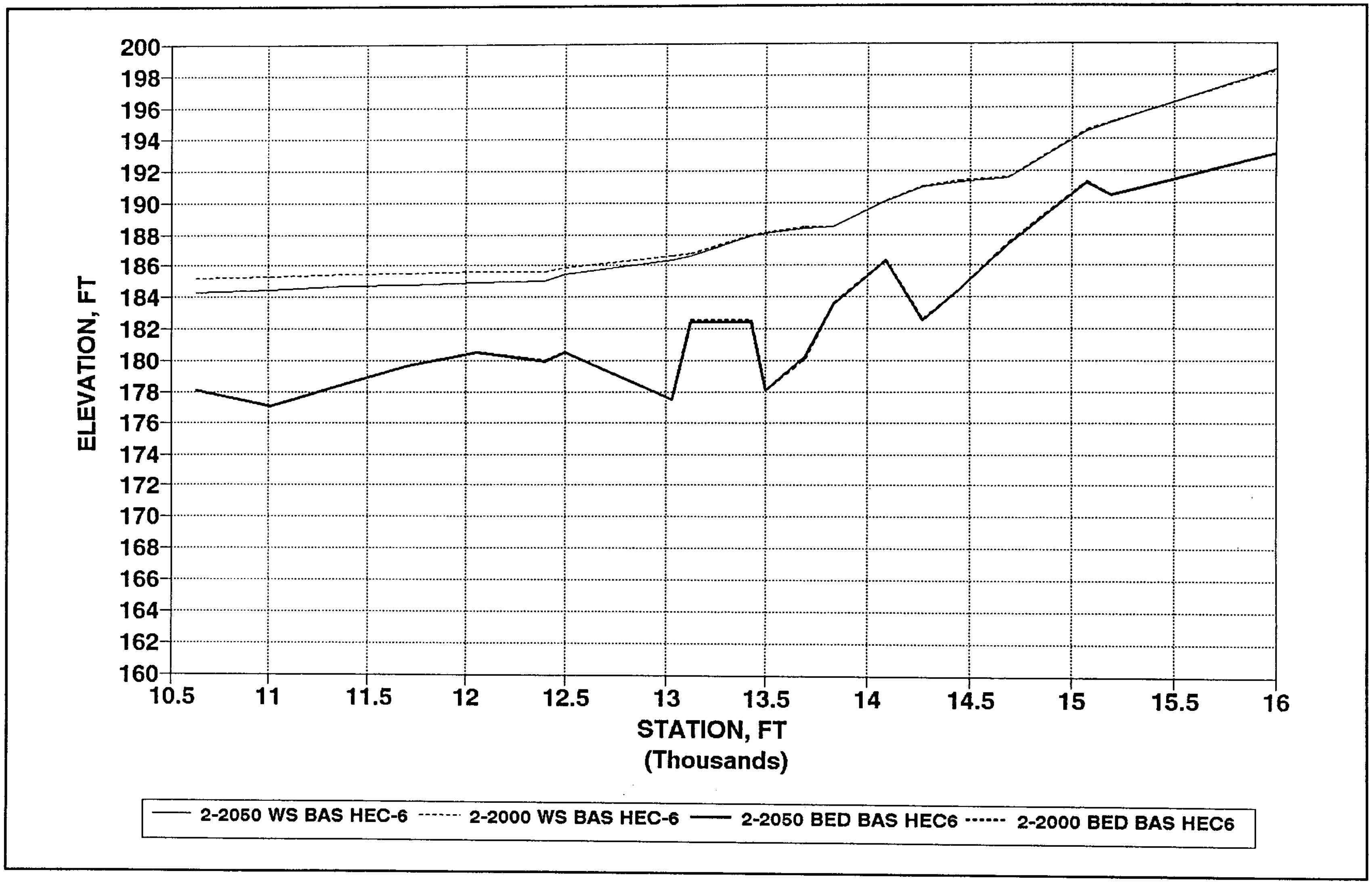


Figure C15. Base long-term simulation, bed and water-surface profiles, Branch 12, Pequannock River

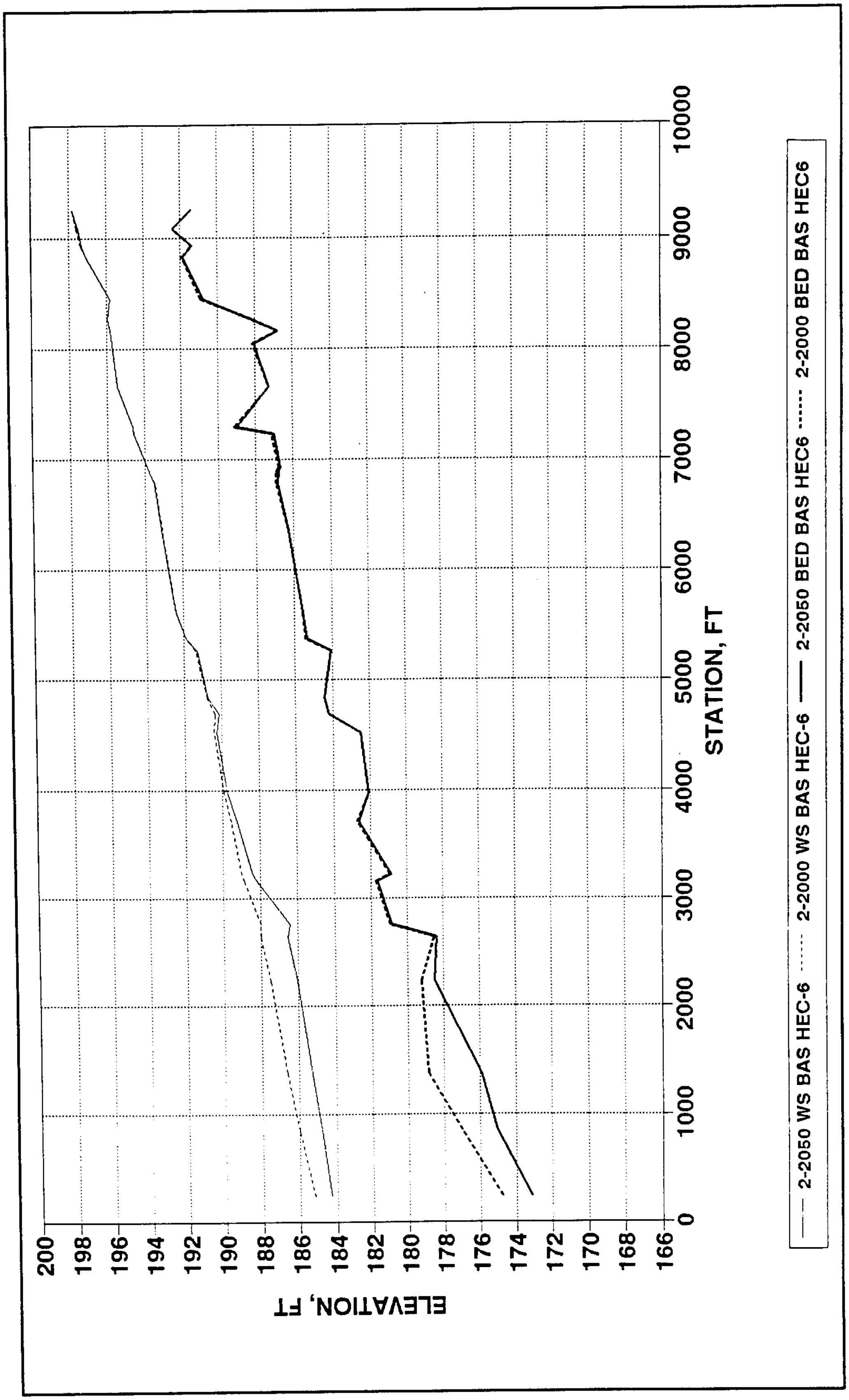


Figure C16. Base long-term simulation, bed and water-surface profiles, Branch 13, Wanaque River

Appendix D
Plan Long-Term Simulation

This appendix contains graphs for river branches 1 to 13 showing the differences between plan beds and plan water surfaces from the initial condition (year 2000) to the year 2050 using a 2-year steady state peak flow. To convert feet to meters, multiply by 0.3048.

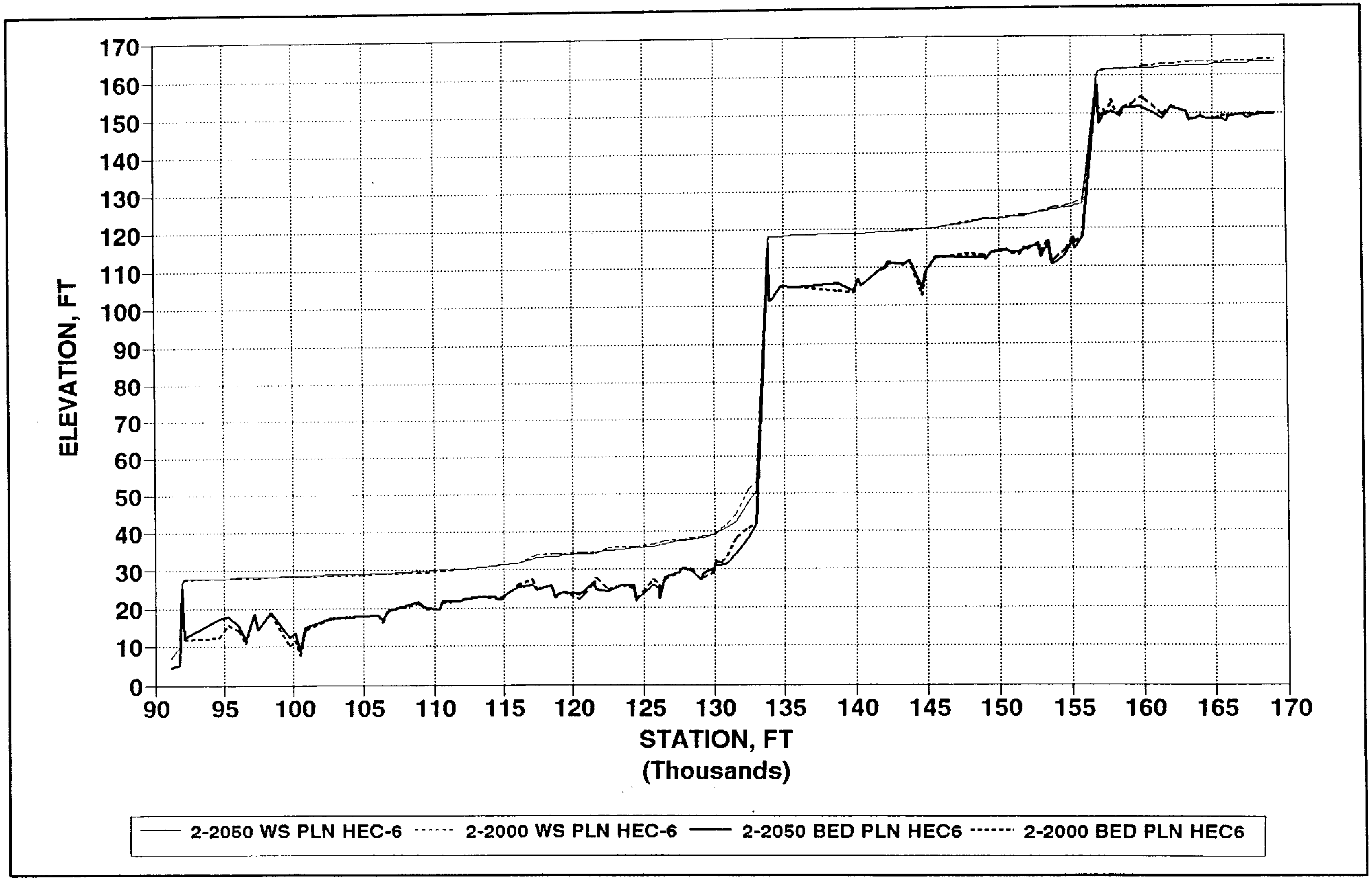


Figure D1. Plan long-term simulation, bed and water-surface profiles, Branch 1, Passaic River

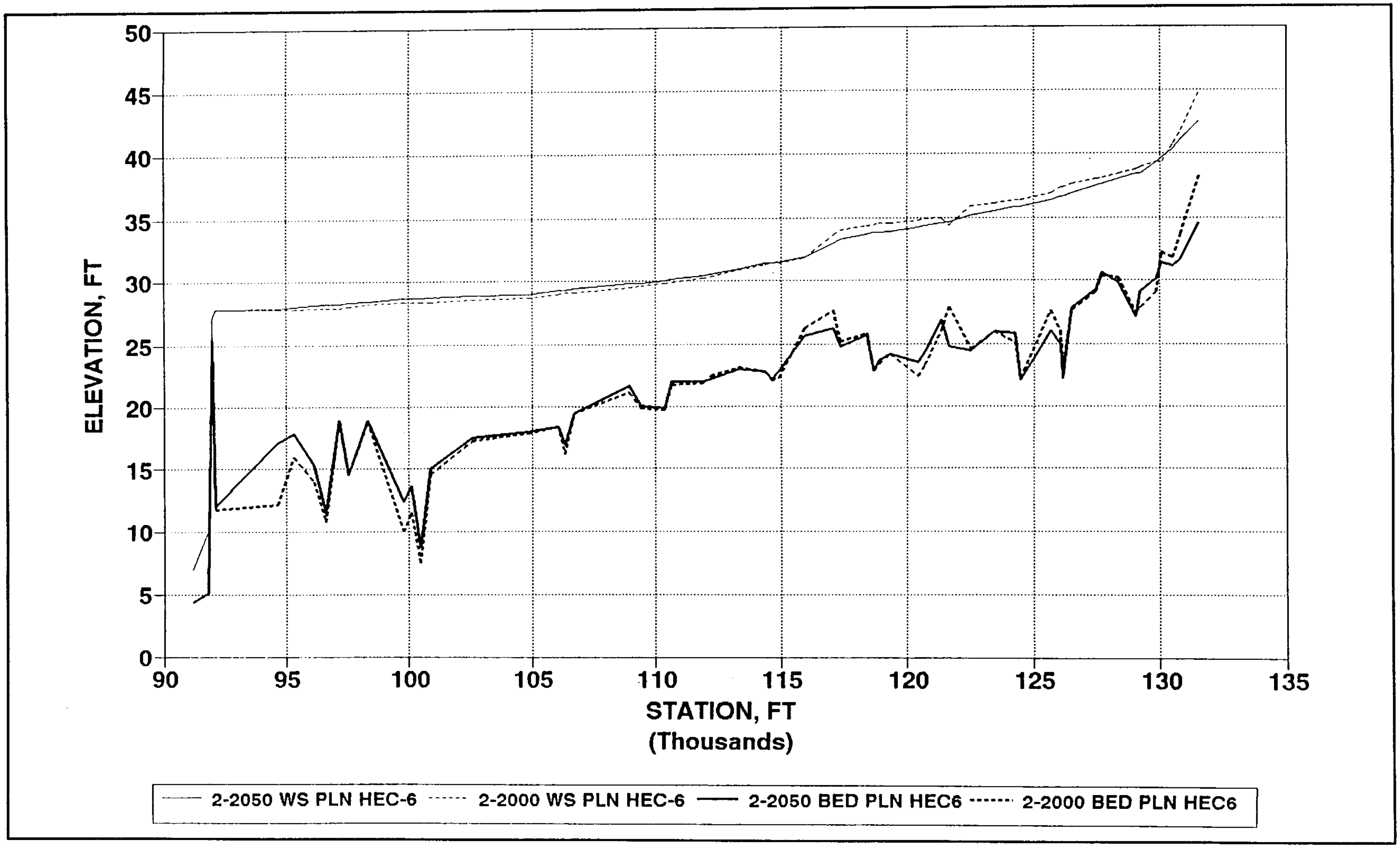


Figure D2. Plan long-term simulation, bed and water-surface profiles, Branch 1, Passaic River, Dundee Dam

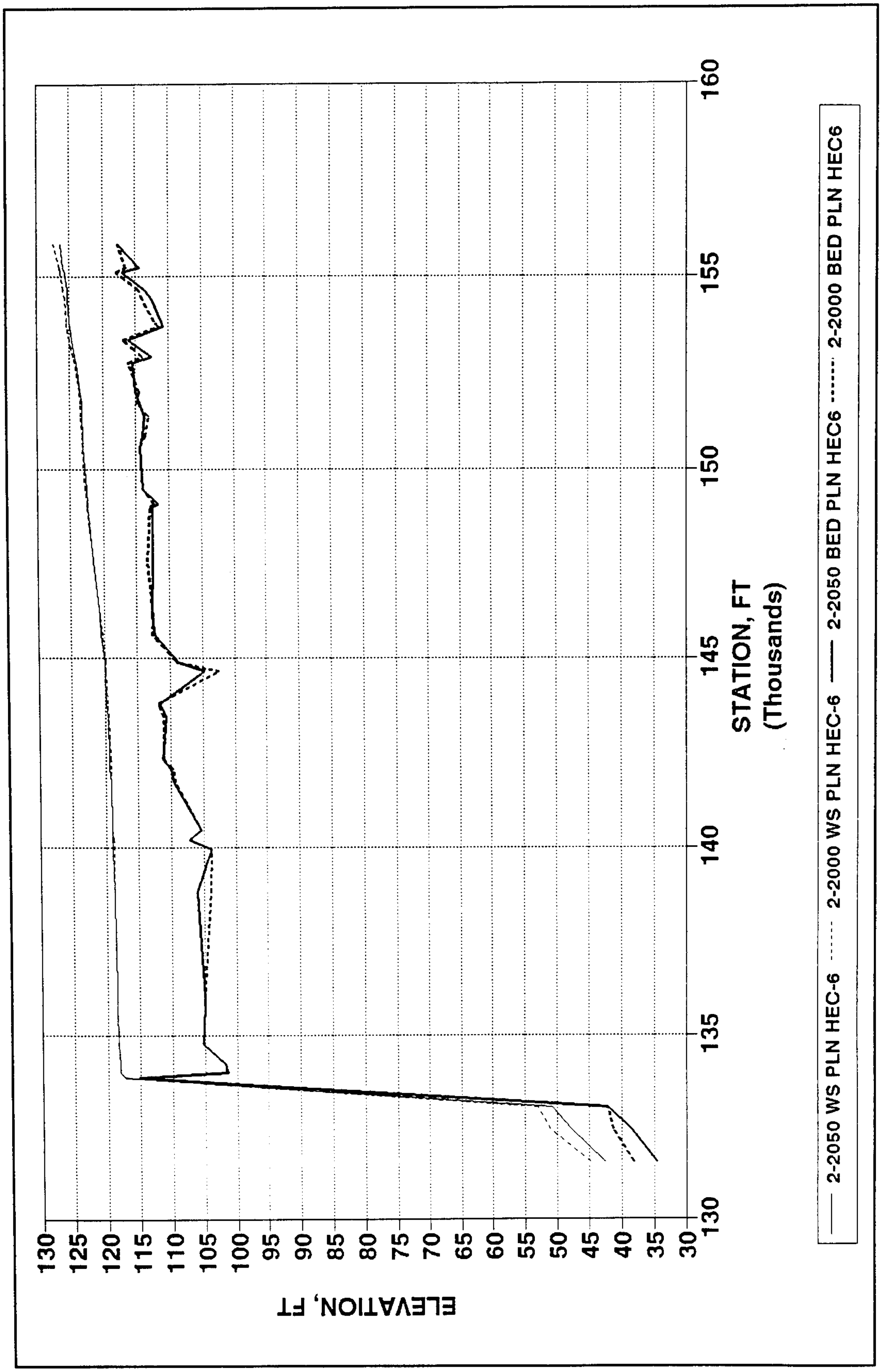


Figure D3. Plan long-term simulation, bed and water-surface profiles, Branch 1, Passaic River, S.U.M. Dam

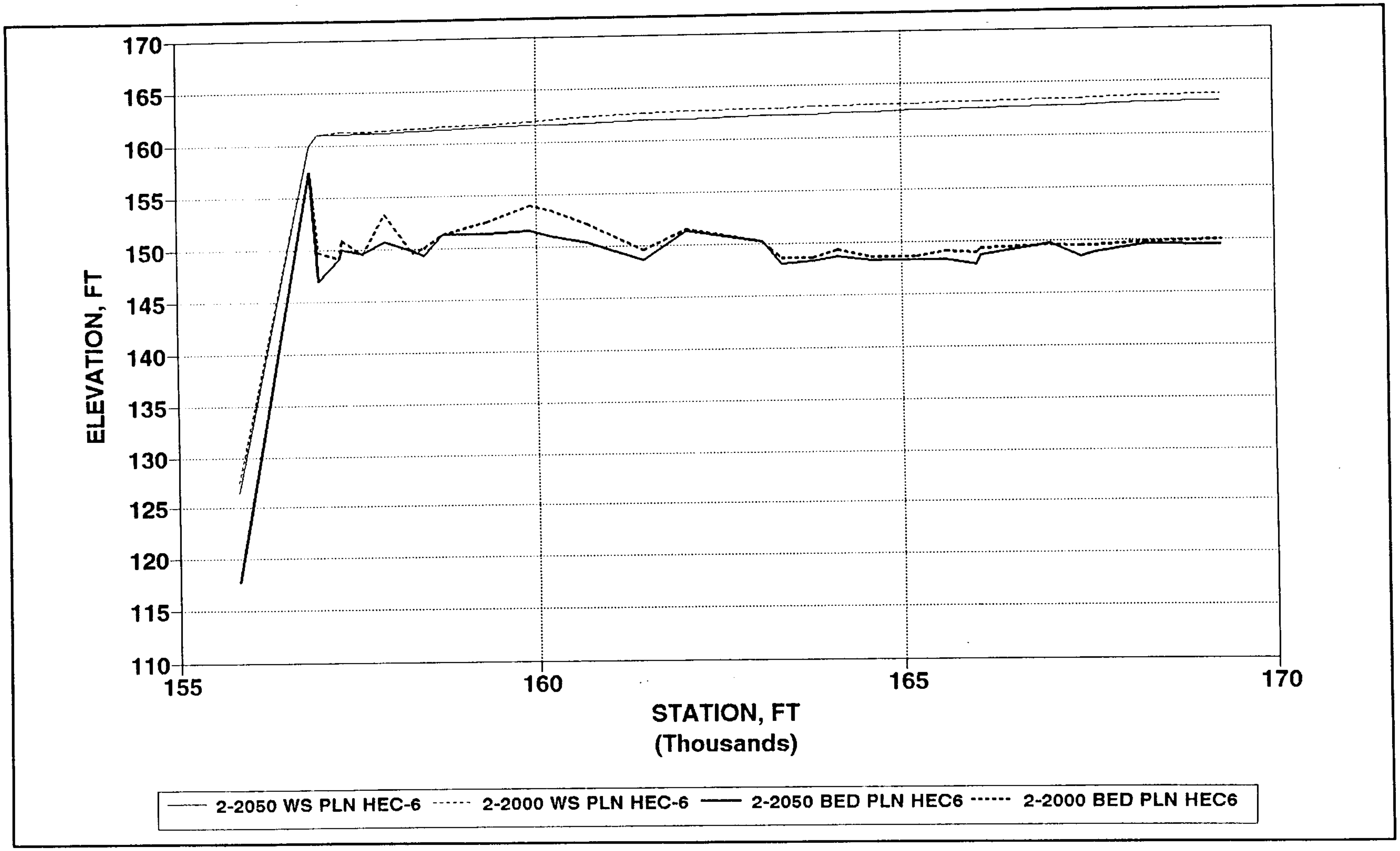


Figure D4. Plan long-term simulation, bed and water-surface profiles, Branch 1, Passaic River, Beatties Dam

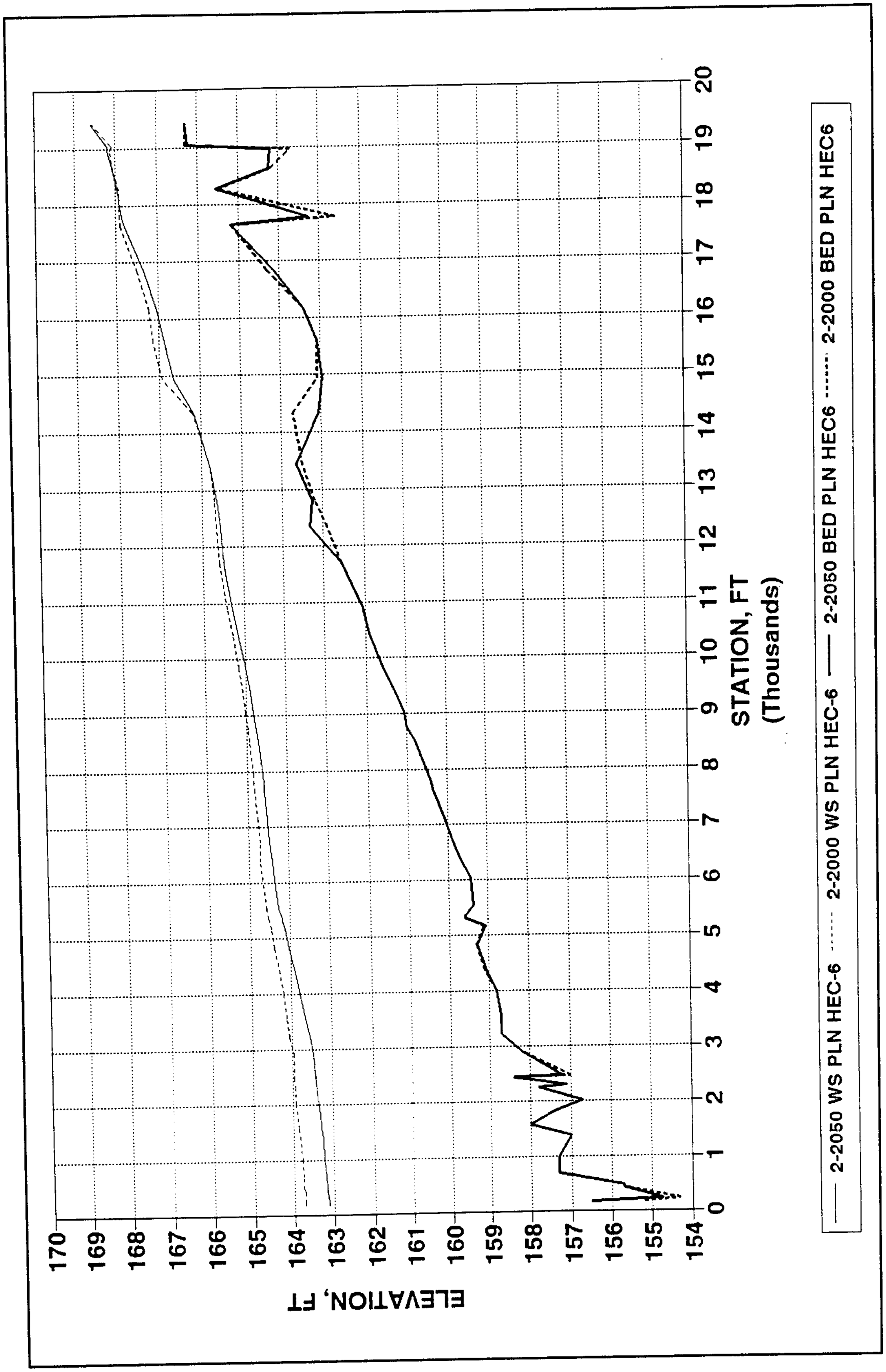


Figure D5. Plan long-term simulation, bed and water-surface profiles, Branch 2, Deepavaal Brook

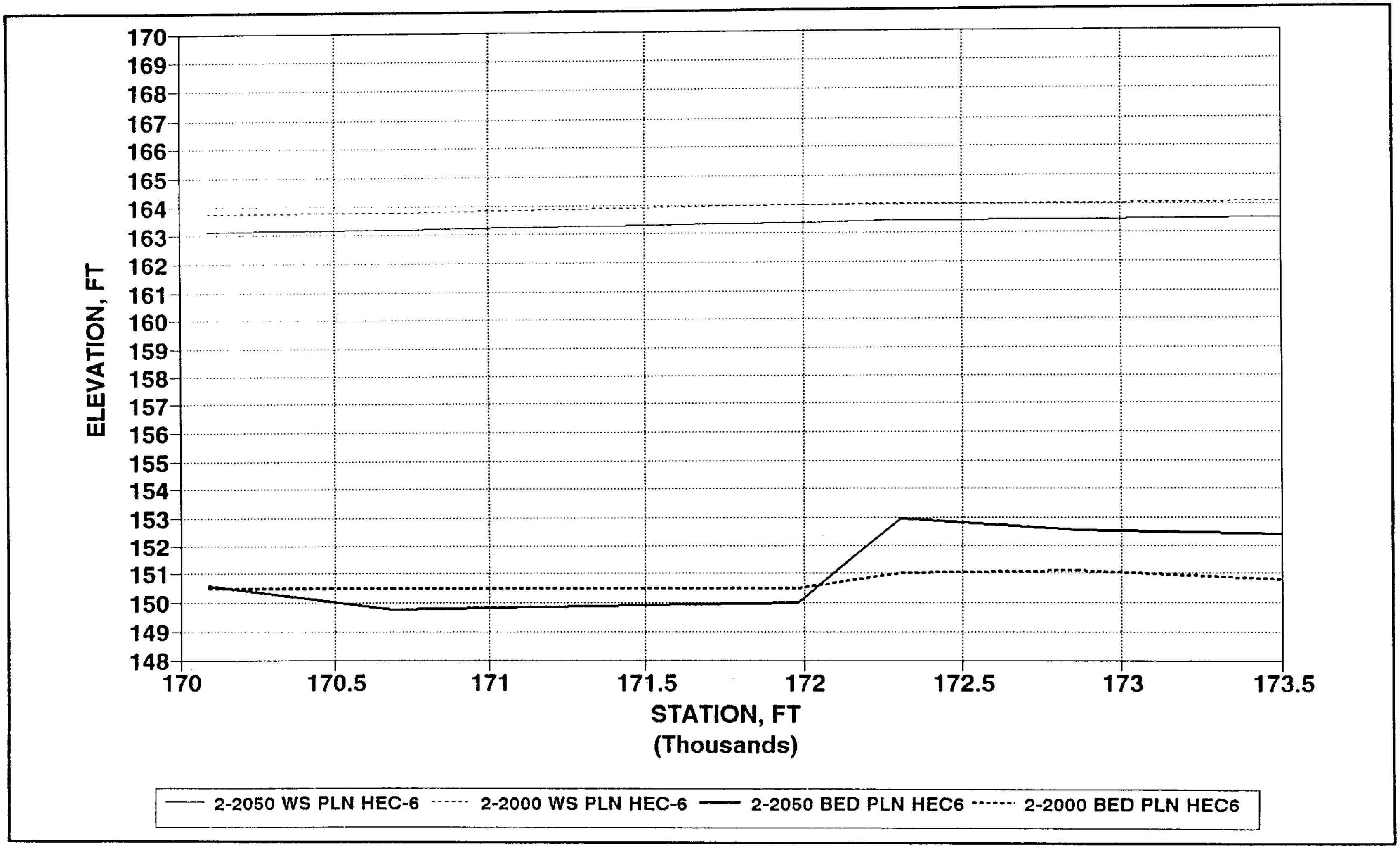


Figure D6. Plan long-term simulation, bed and water-surface profiles, Branch 3, Passaic River

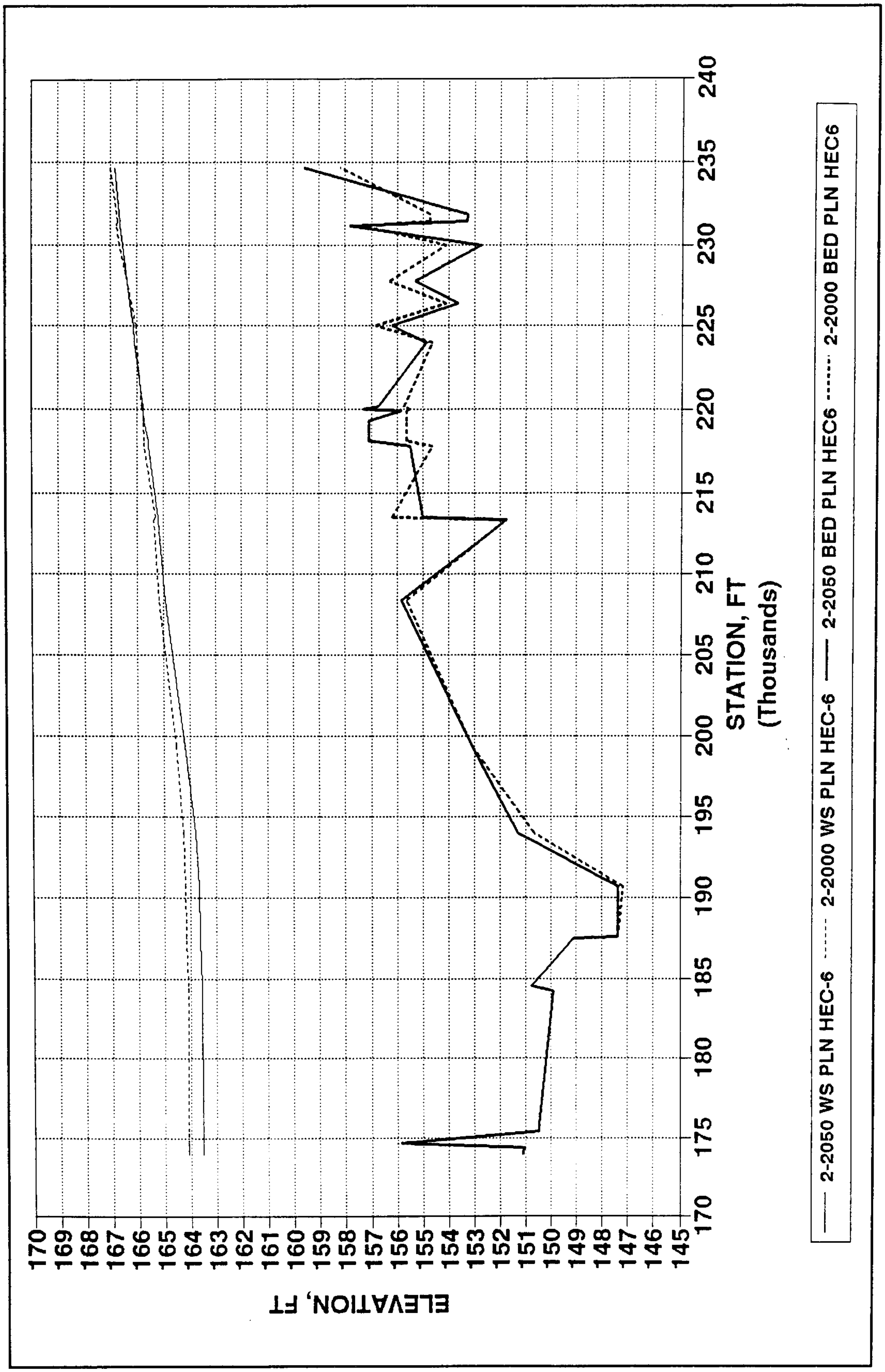


Figure D7. Plan long-term simulation, bed and water-surface profiles, Branch 4, Passaic River

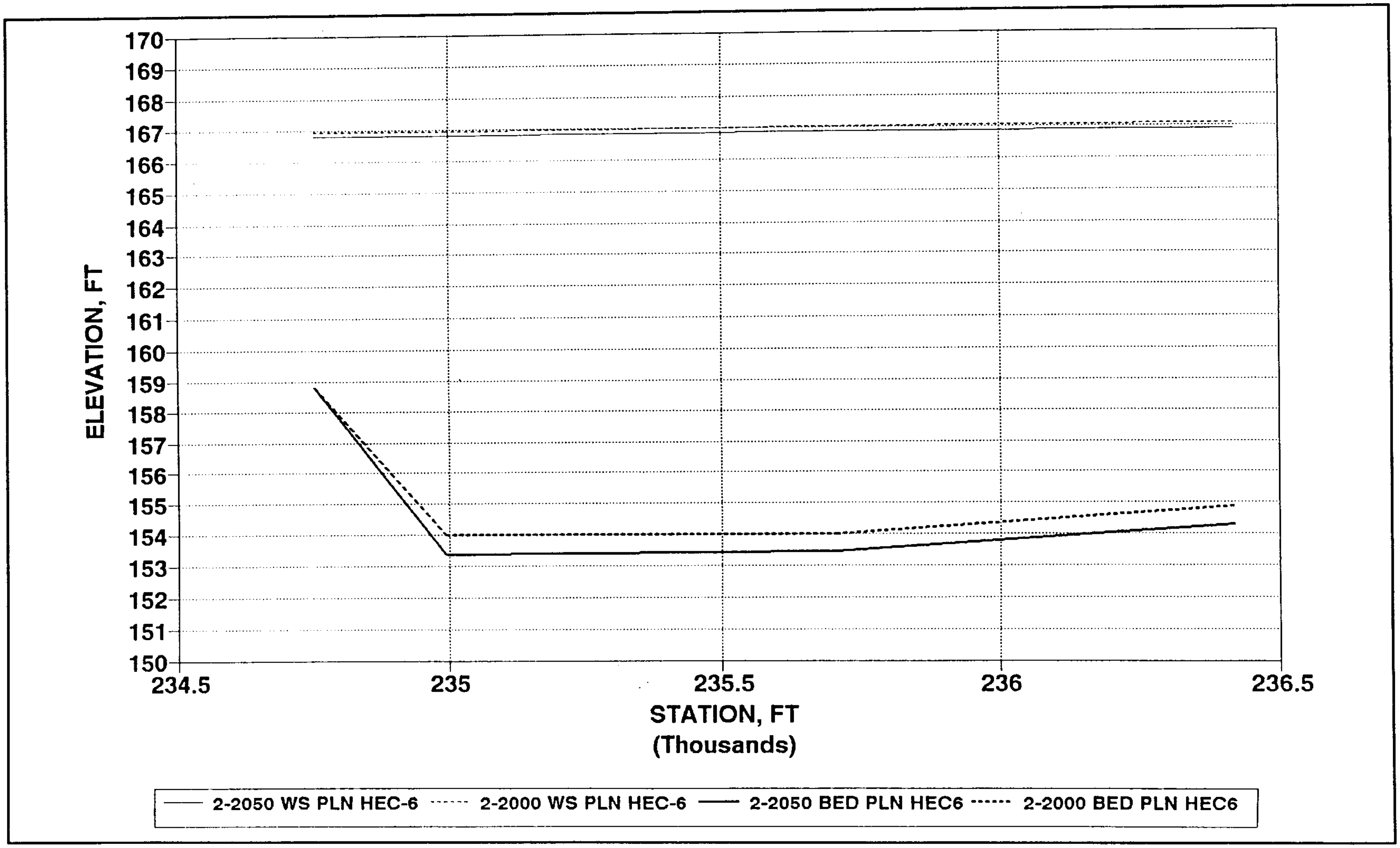


Figure D8. Plan long-term simulation, bed and water-surface profiles, Branch 5, Passaic River

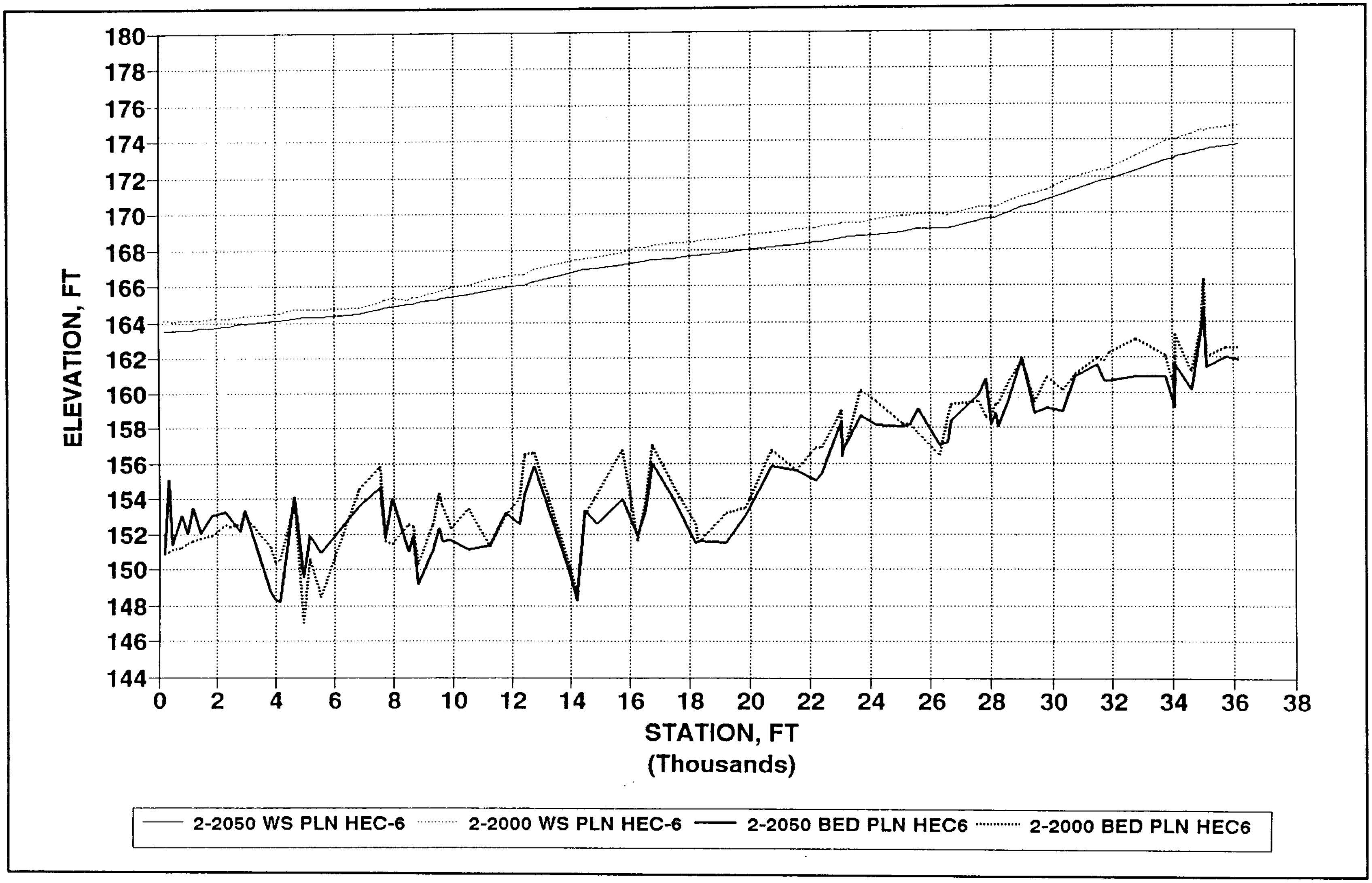


Figure D9. Plan long-term simulation, bed and water-surface profiles, Branch 6, Pompton River

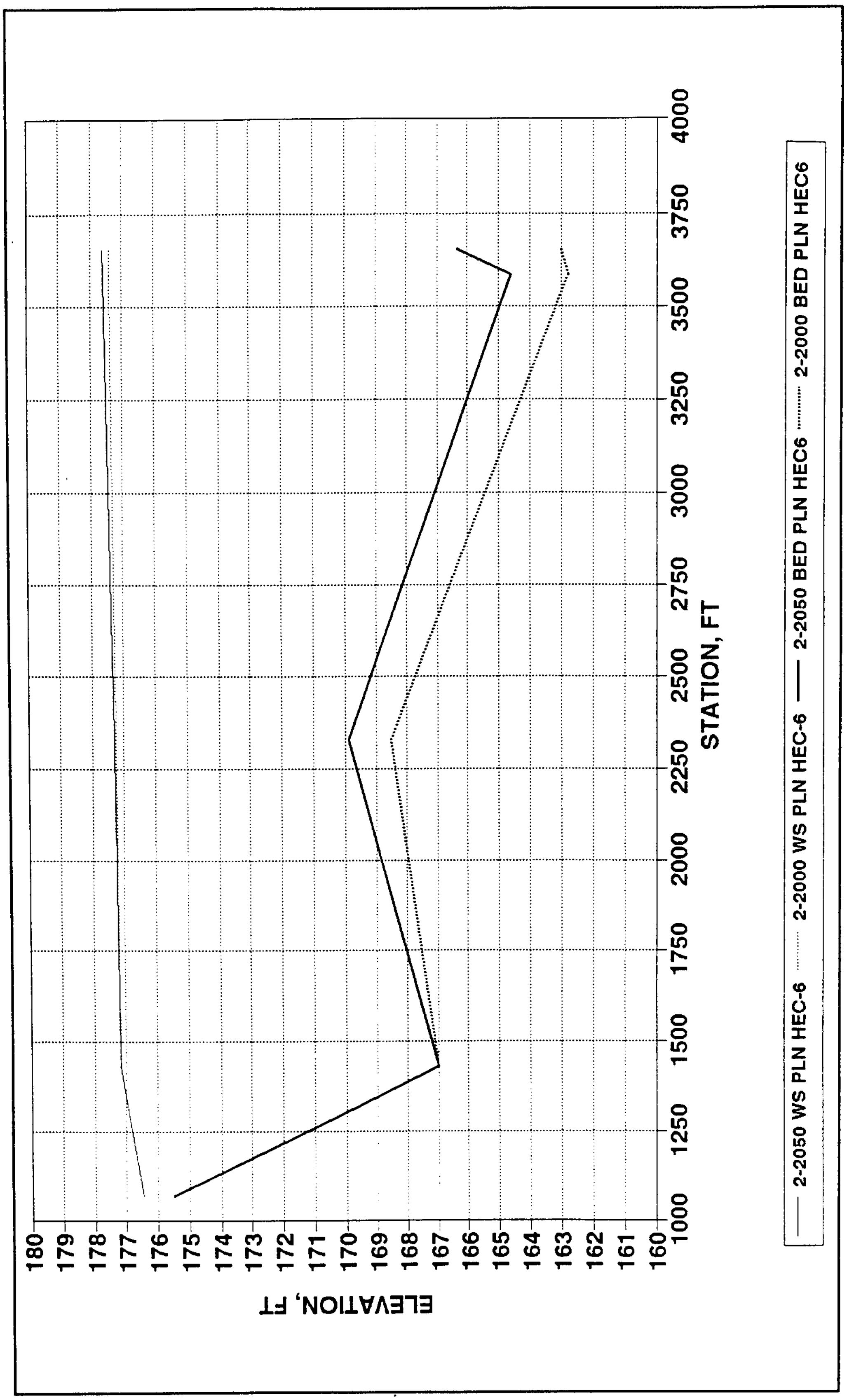


Figure D10. Plan long-term simulation, bed and water-surface profiles, Branch 7, Ramapo River

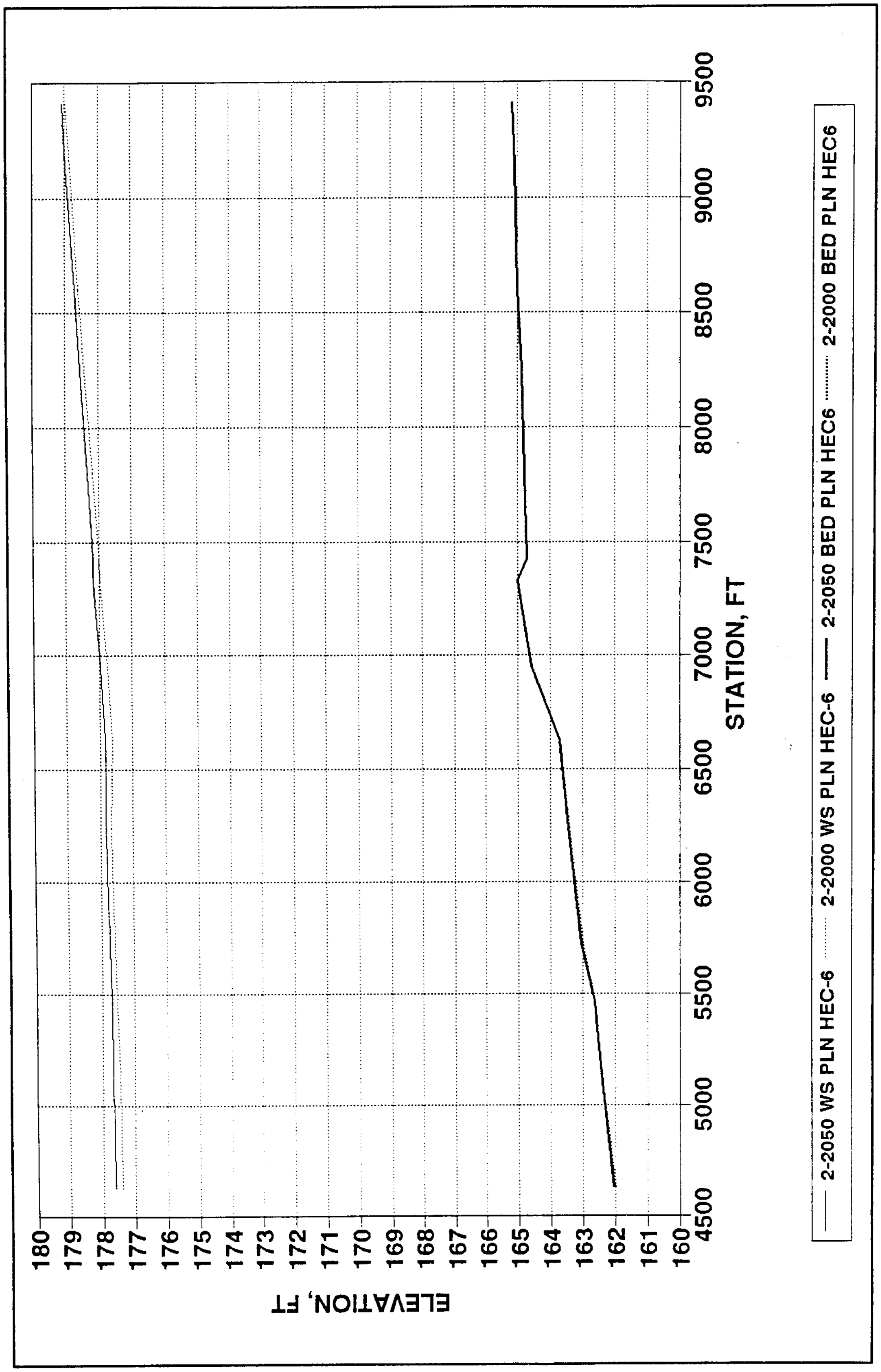


Figure D11. Plan long-term simulation, bed and water-surface profiles, Branch 8, Ramapo River

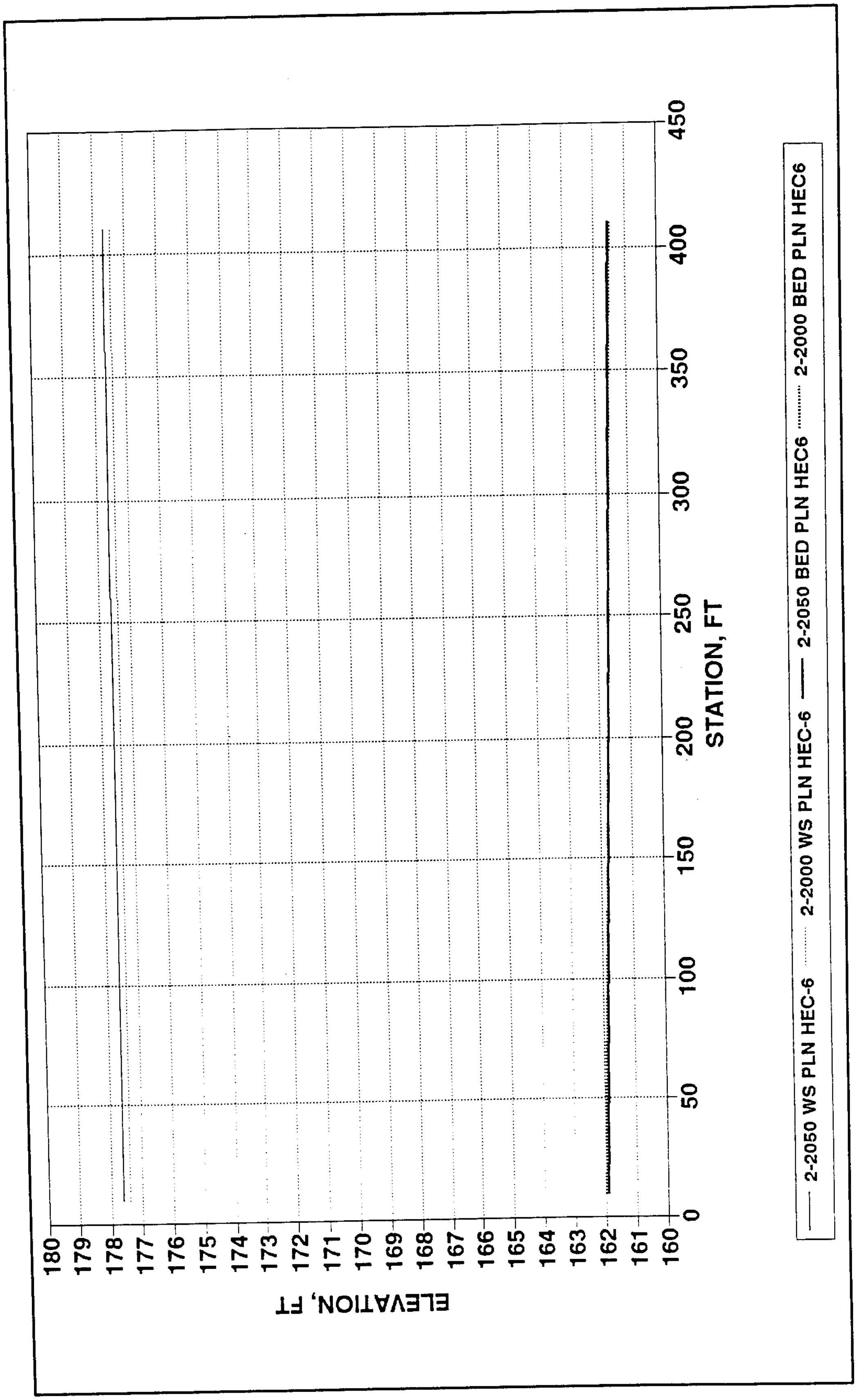


Figure D12. Plan long-term simulation, bed and water-surface profiles, Branch 9

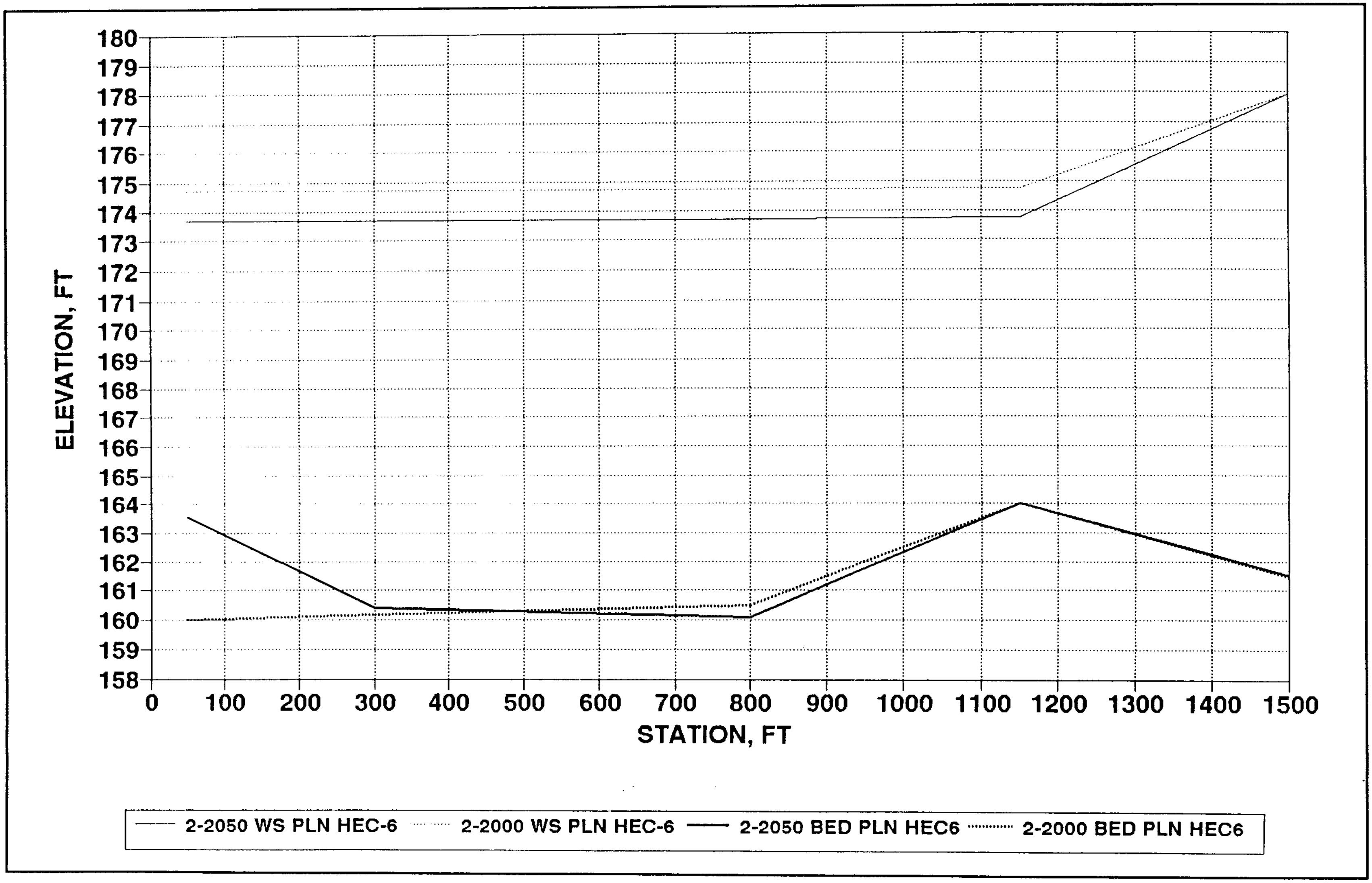


Figure D13. Plan long-term simulation, bed and water-surface profiles, Branch 10, Bypass

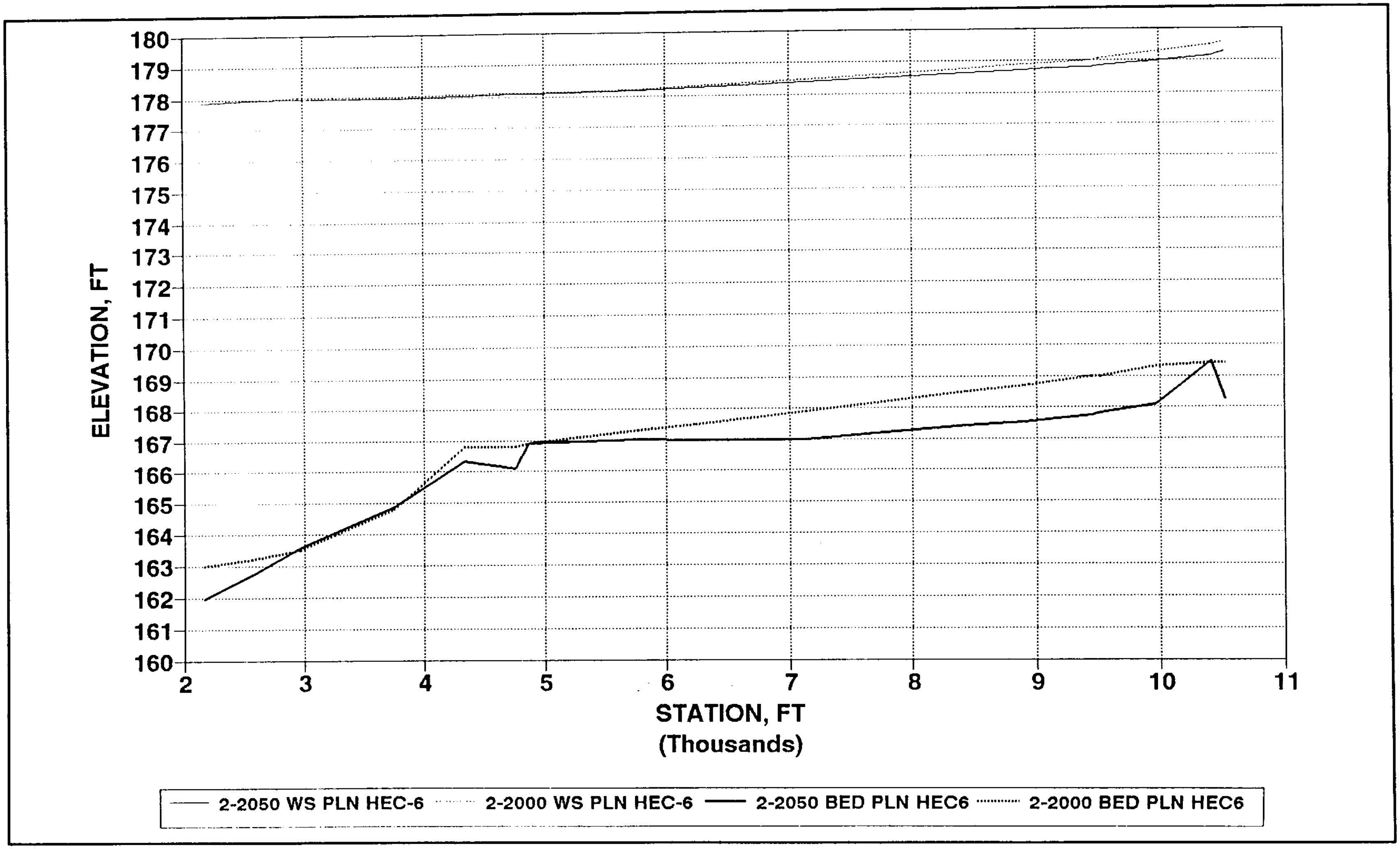


Figure D14. Plan long-term simulation, bed and water-surface profiles, Branch 11, Pequannock River

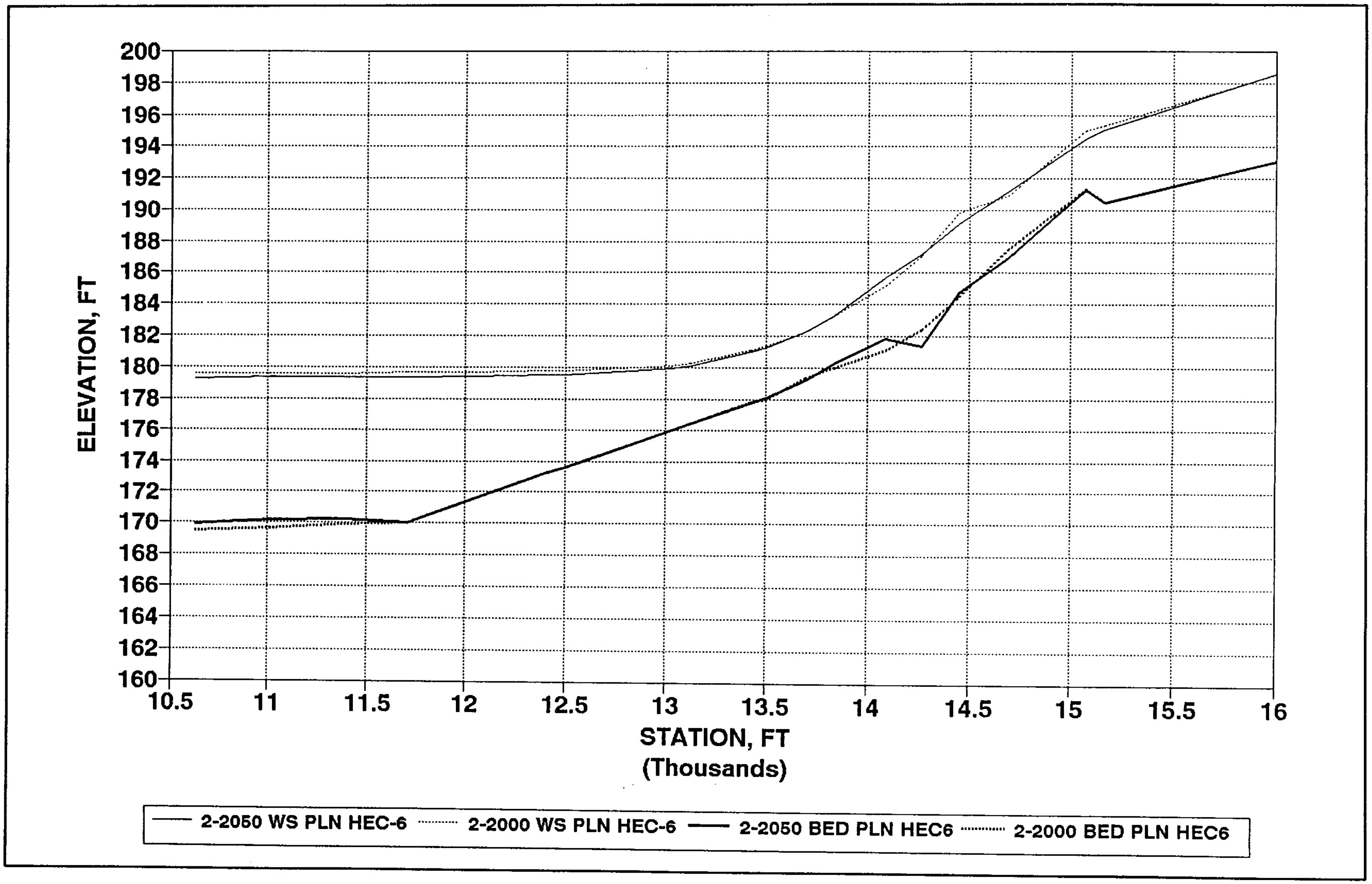


Figure D15. Plan long-term simulation, bed and water-surface profiles, Branch 12, Pequannock River

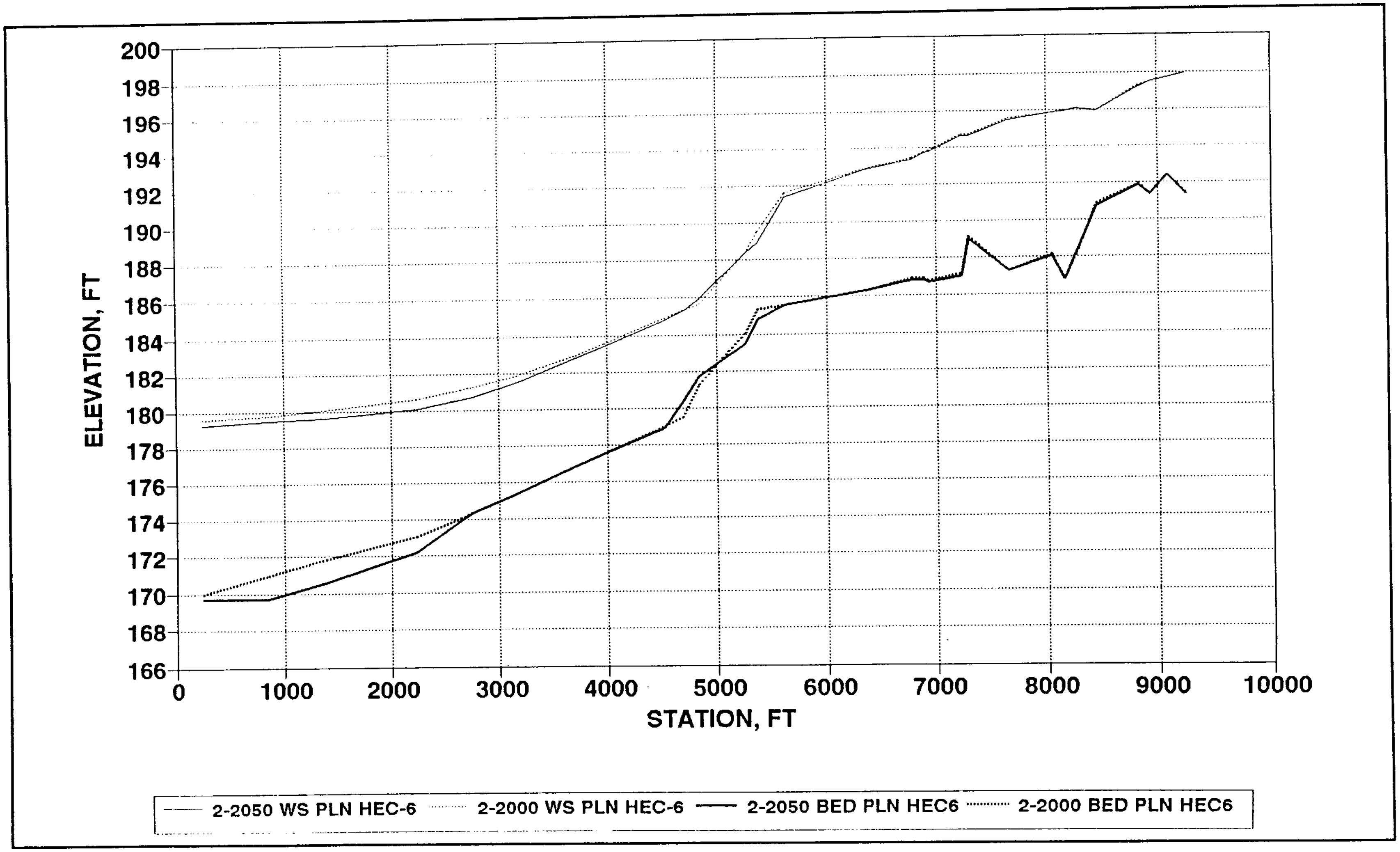


Figure D16. Plan long-term simulation, bed and water-surface profiles, Branch 13, Wanaque River

Appendix E
Base Versus Plan Initial Condition

This appendix contains graphs for river branches 1 to 13 showing the differences between base and plan beds and base and plan water surfaces at the initial condition (year 2000) and using a 2-year steady state peak flow. To convert feet to meters, multiply by 0.3048.

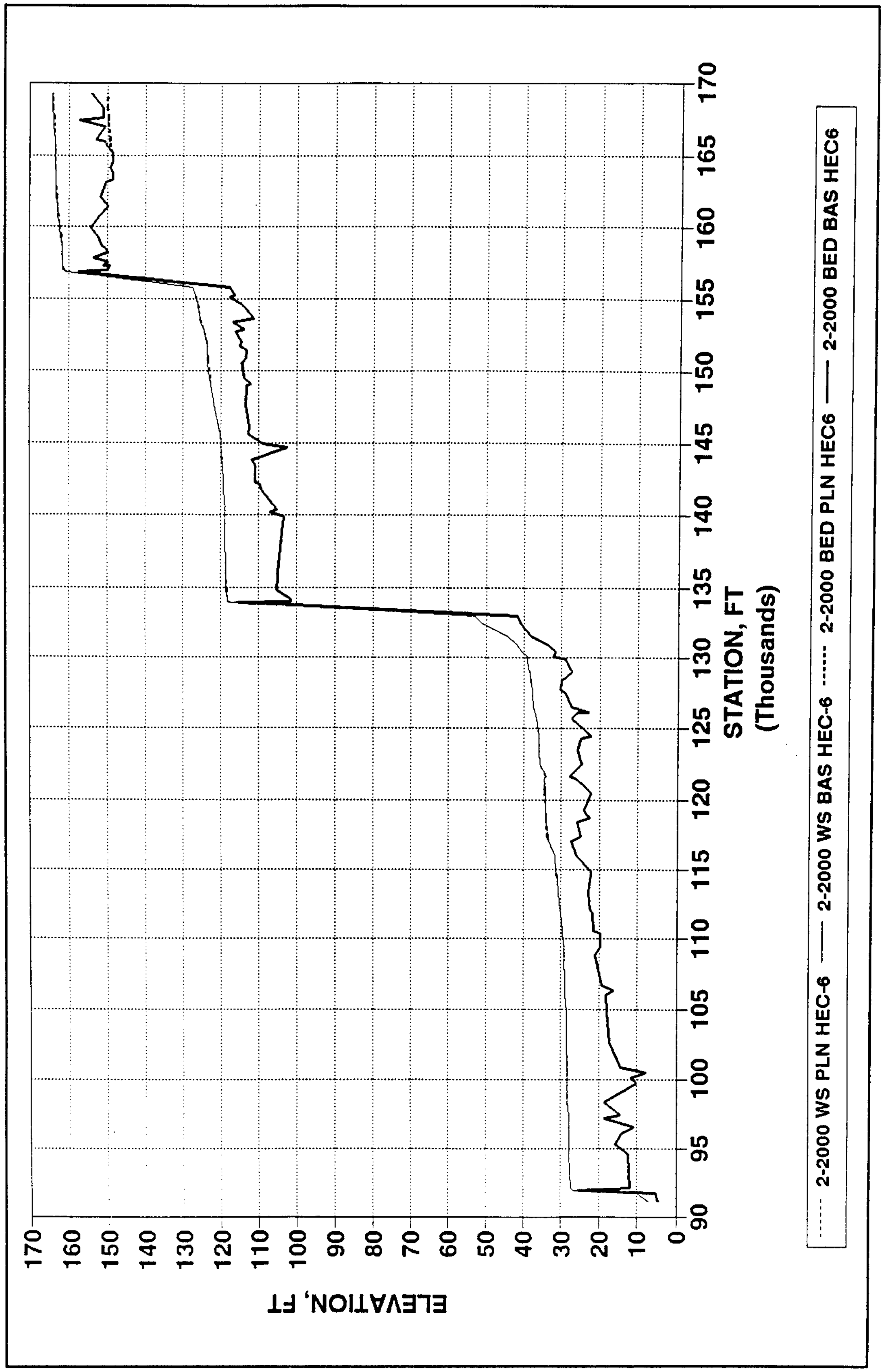


Figure E1. Base versus plan initial condition, bed and water-surface profiles, Branch 1, Passaic River

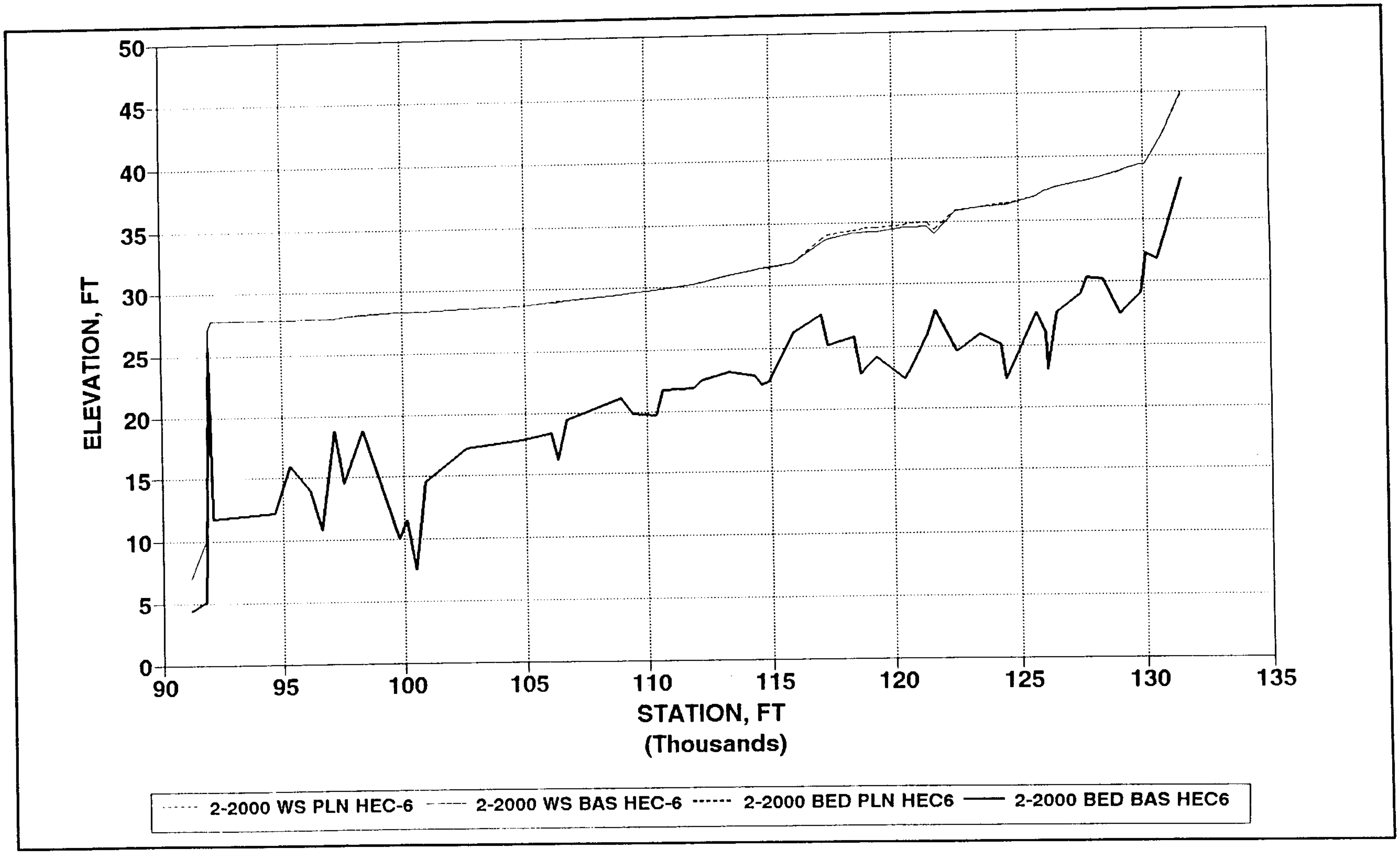


Figure E2. Base versus plan initial condition, bed and water-surface profiles, Branch 1, Passaic River, Dundee Dam

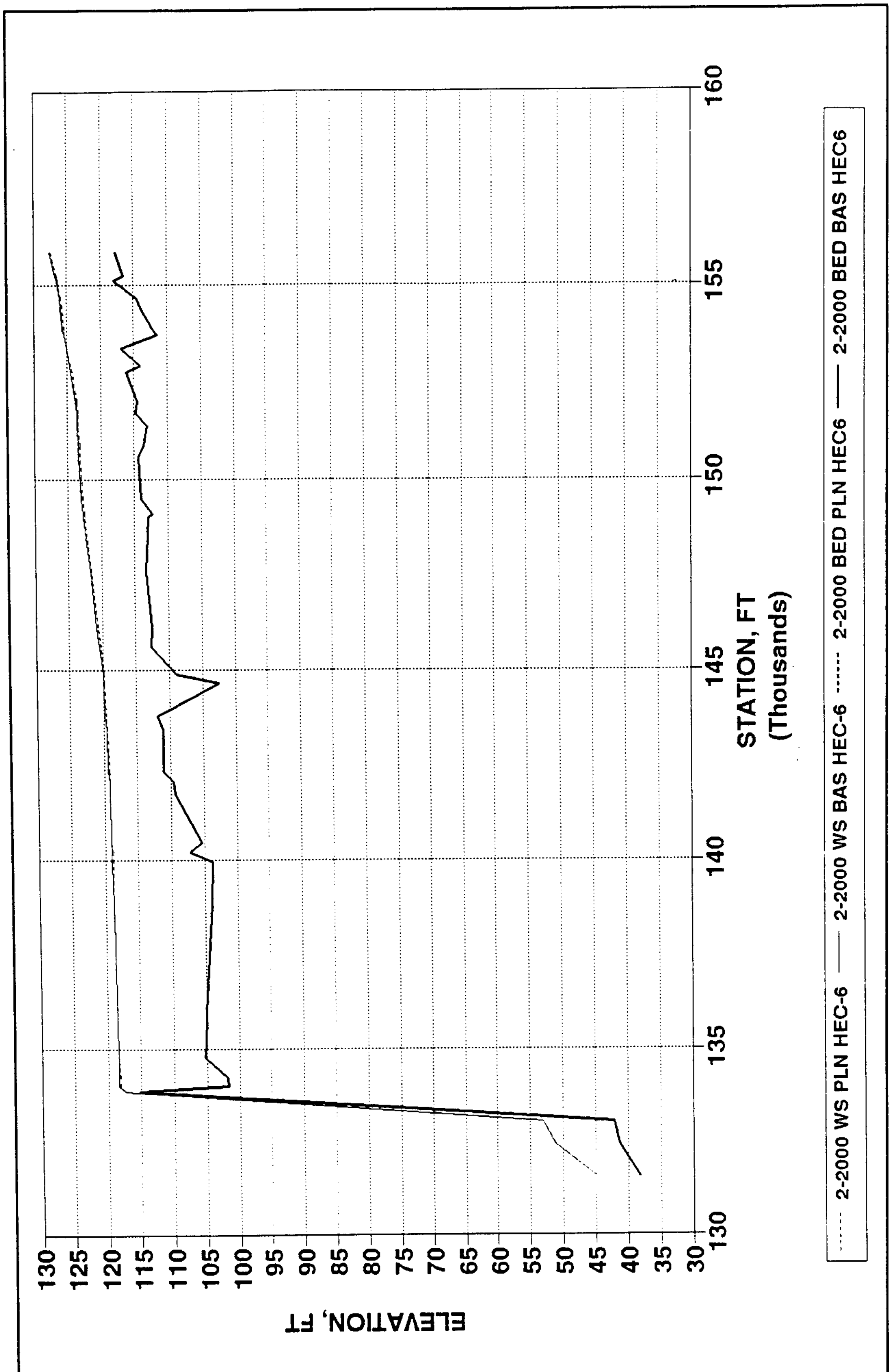


Figure E3. Base versus plan initial condition, bed and water-surface profiles, Branch 1, Passaic River, S.U.M. Dam

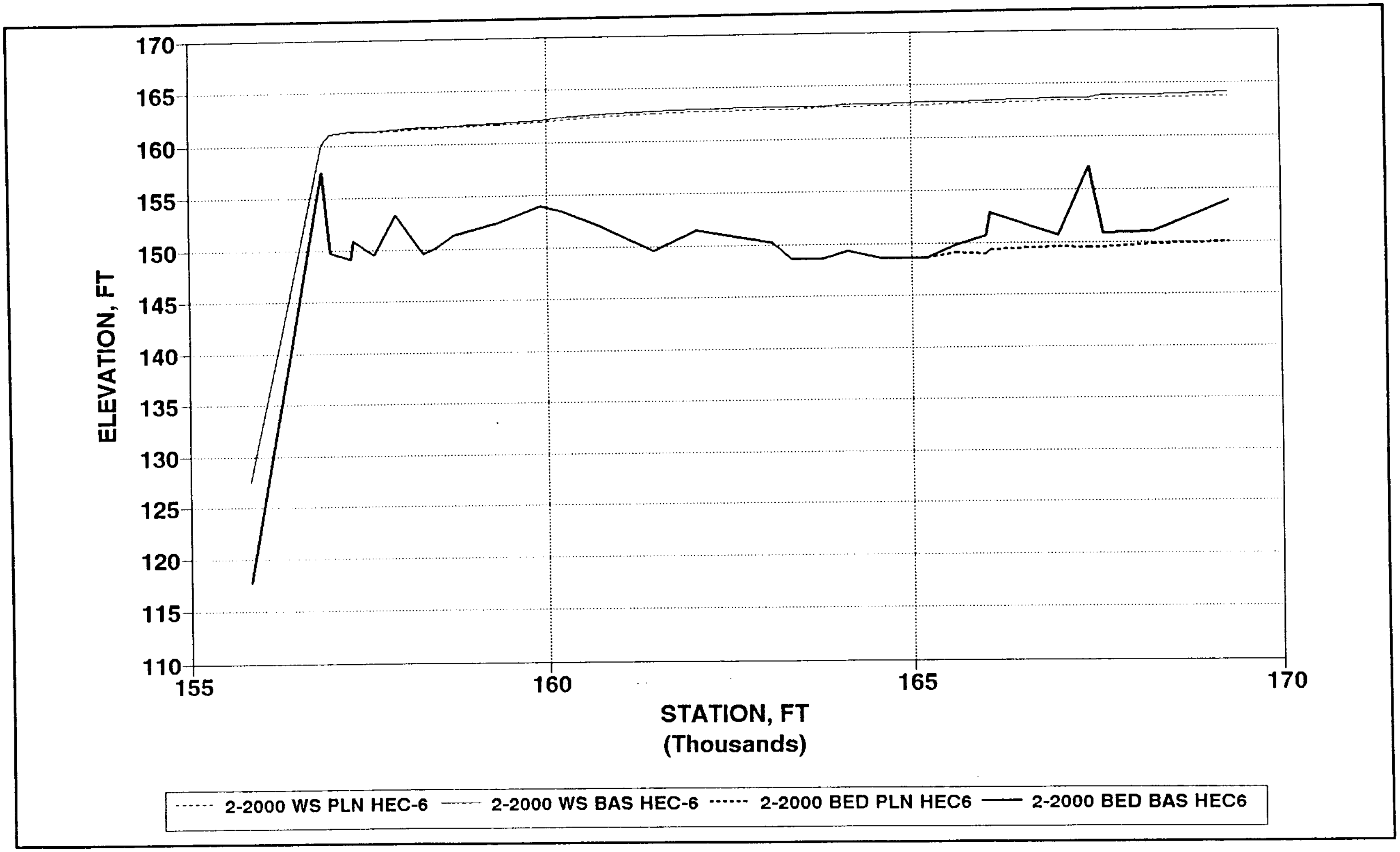


Figure E4. Base versus plan initial condition, bed and water-surface profiles, Branch 1, Passaic River, Beatties Dam

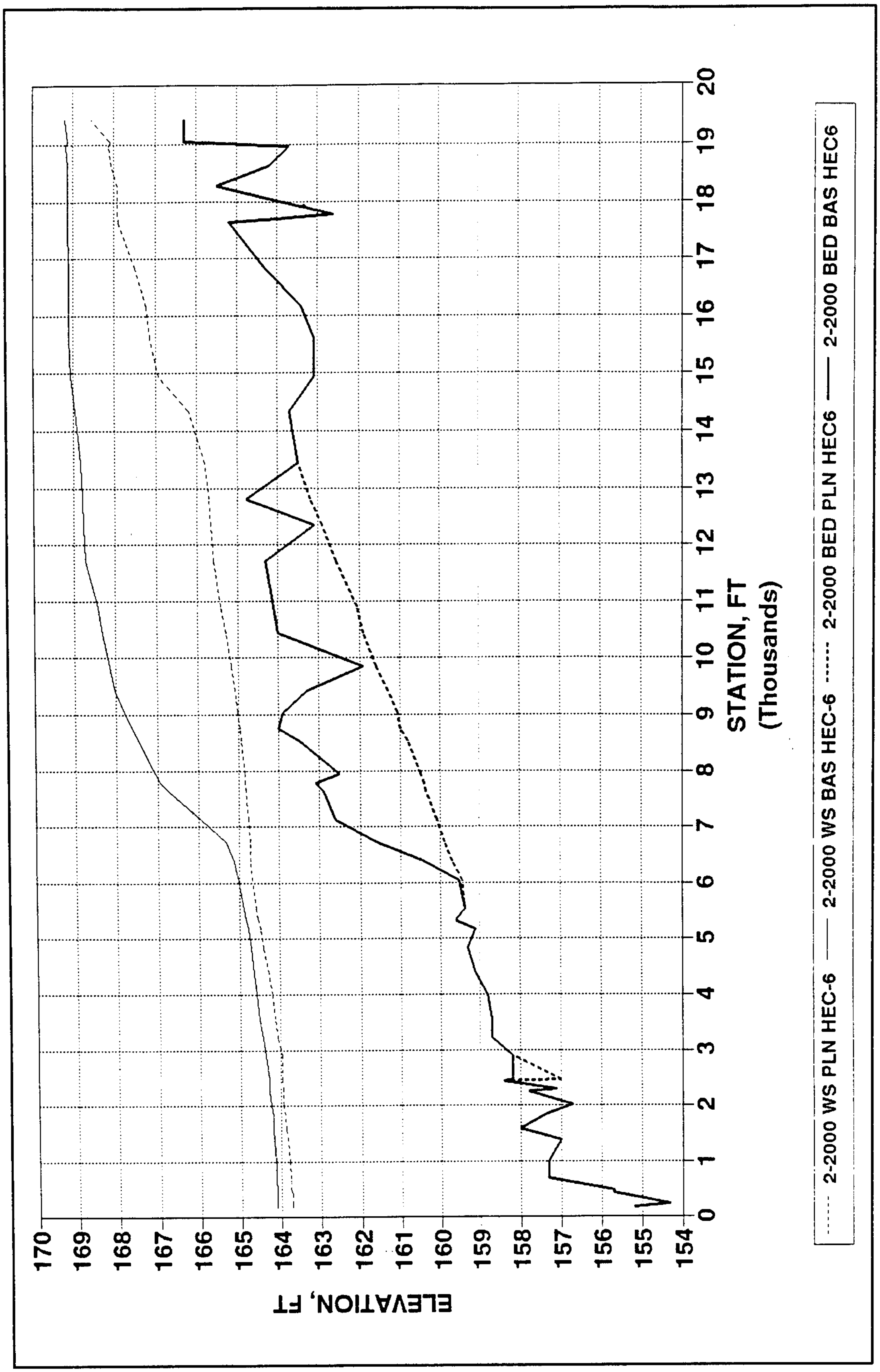


Figure E5. Base versus plan initial condition, bed and water-surface profiles, Branch 2, Deepavaal Brook

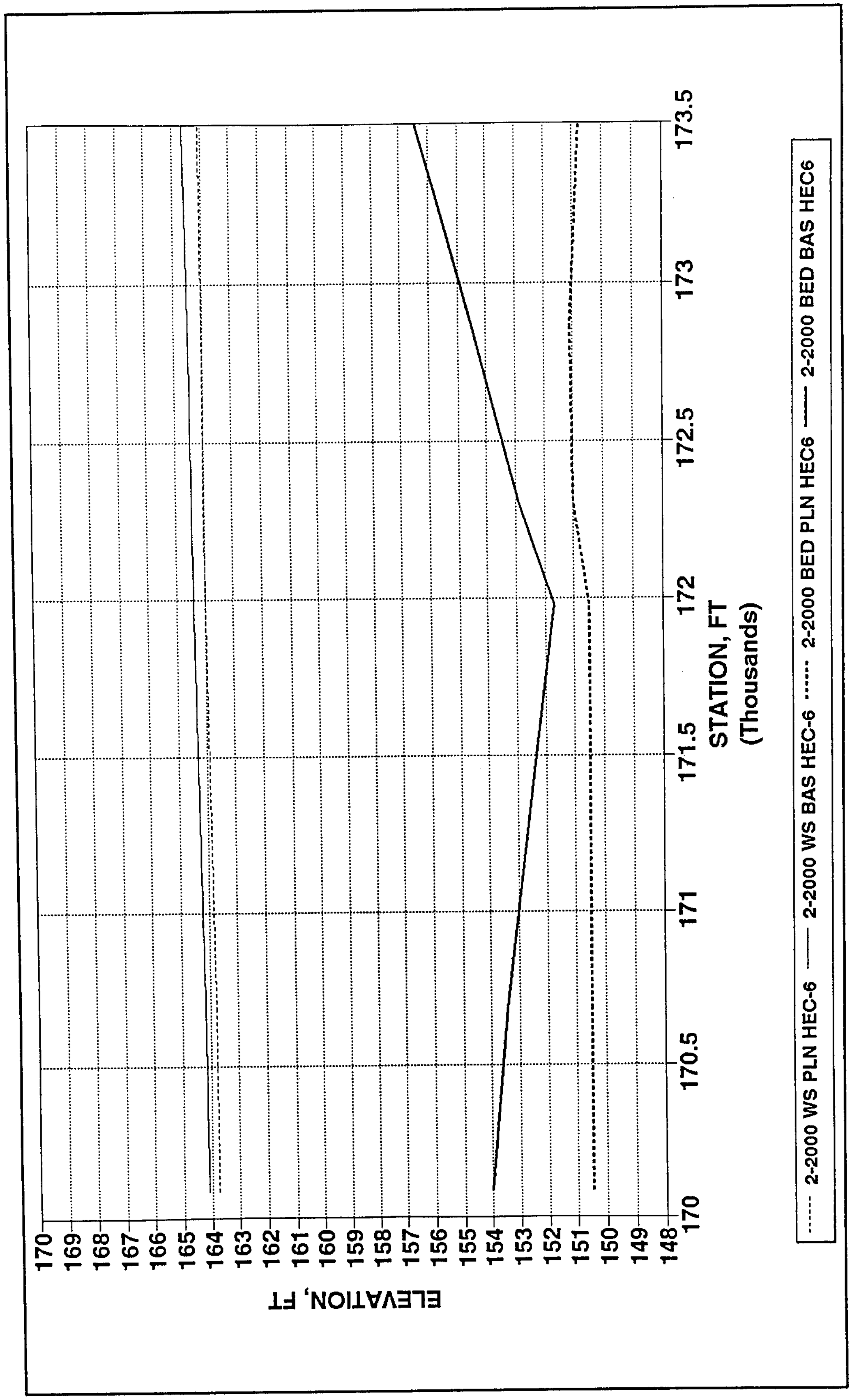


Figure E6. Base versus plan initial condition, bed and water-surface profiles, Branch 3, Passaic River

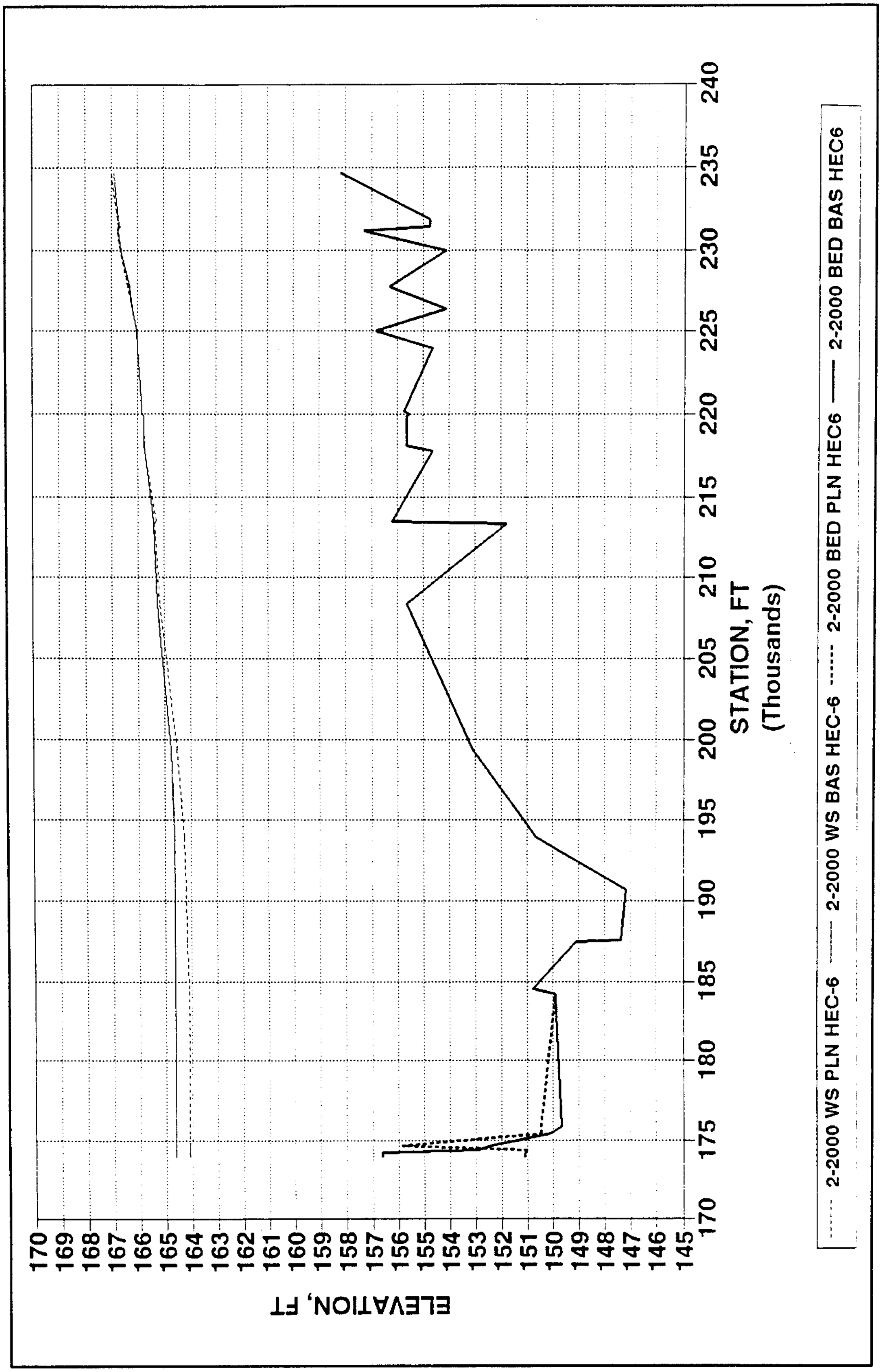


Figure E7. Base versus plan initial condition, bed and water-surface profiles, Branch 4, Passaic River

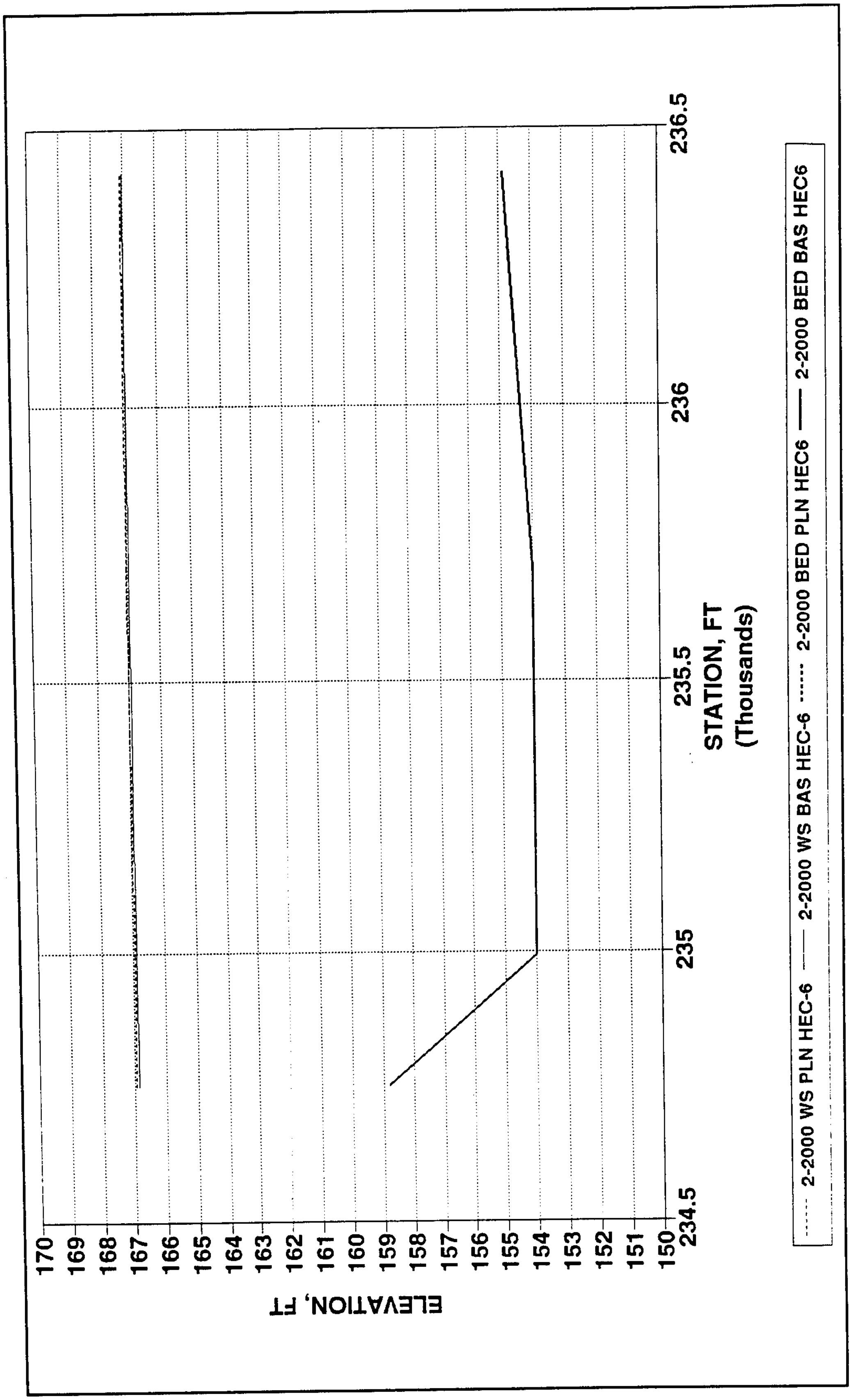


Figure E8. Base versus plan initial condition, bed and water-surface profiles, Branch 5, Passaic River

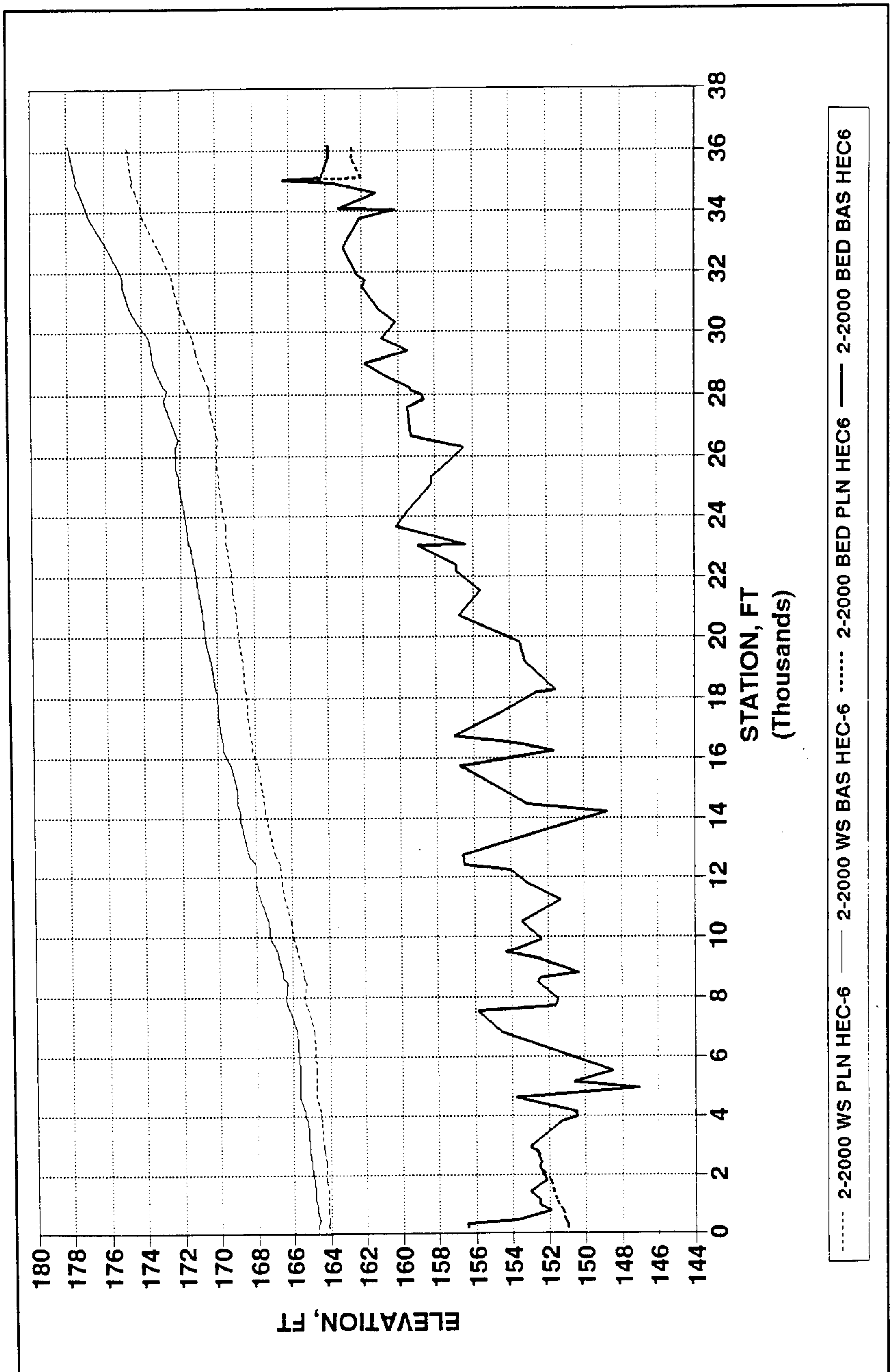


Figure E9. Base versus plan initial condition, bed and water-surface profiles, Branch 6, Pompton River

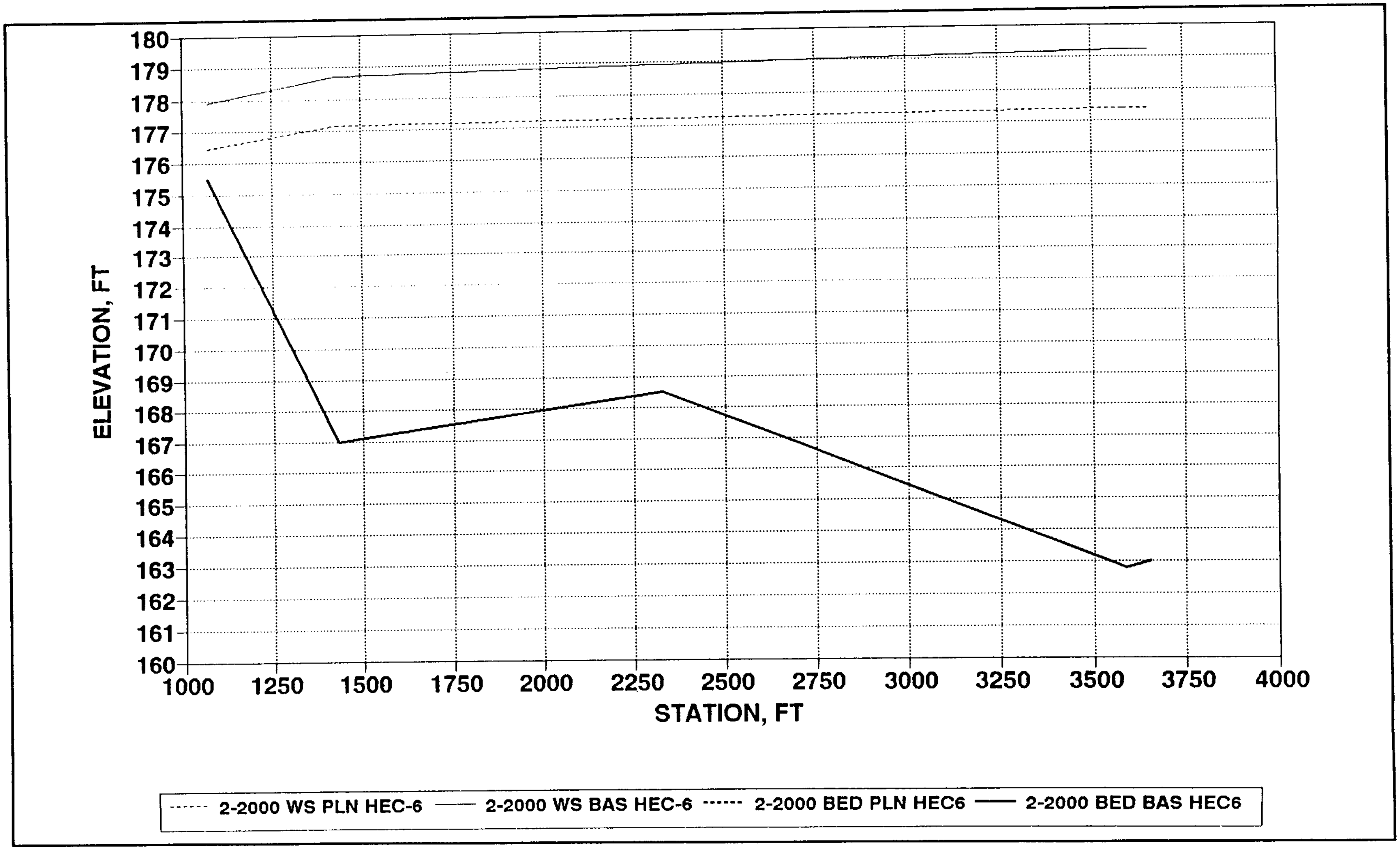


Figure E10. Base versus plan initial condition, bed and water-surface profiles, Branch 7, Ramapo River

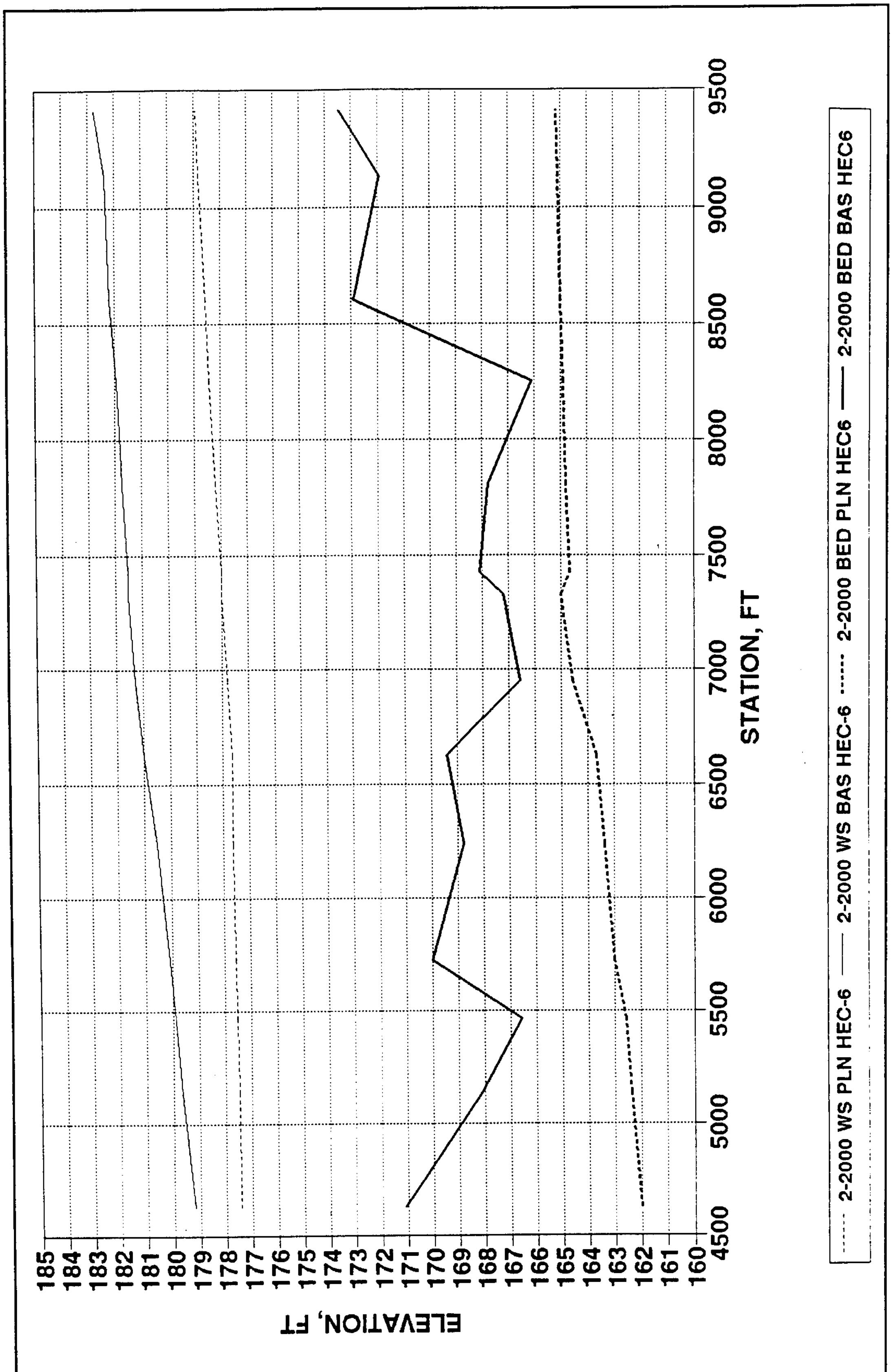


Figure E11. Base versus plan initial condition, bed and water-surface profiles, Branch 8, Ramapo River

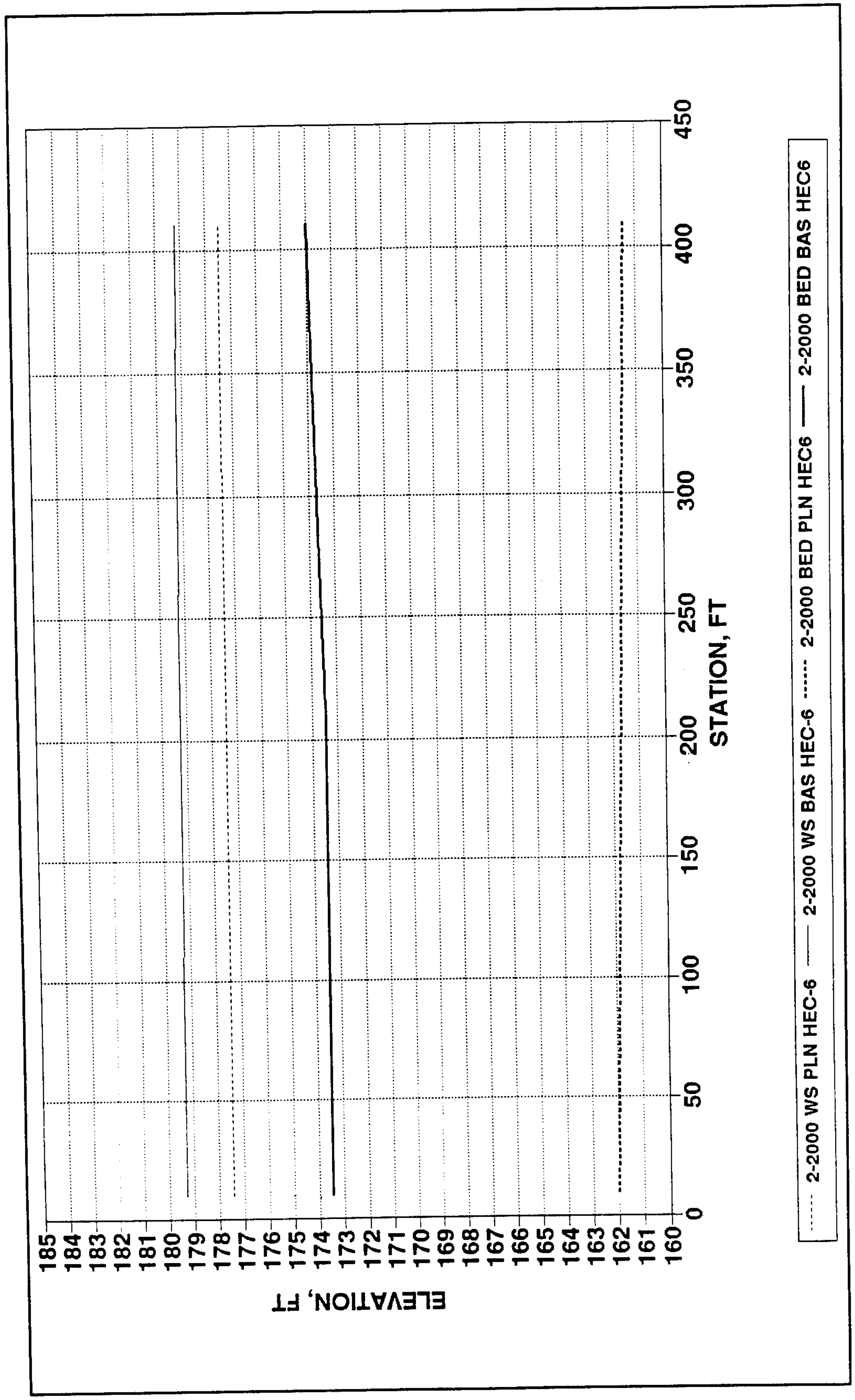


Figure E12. Base versus plan initial condition, bed and water-surface profiles, Branch 9

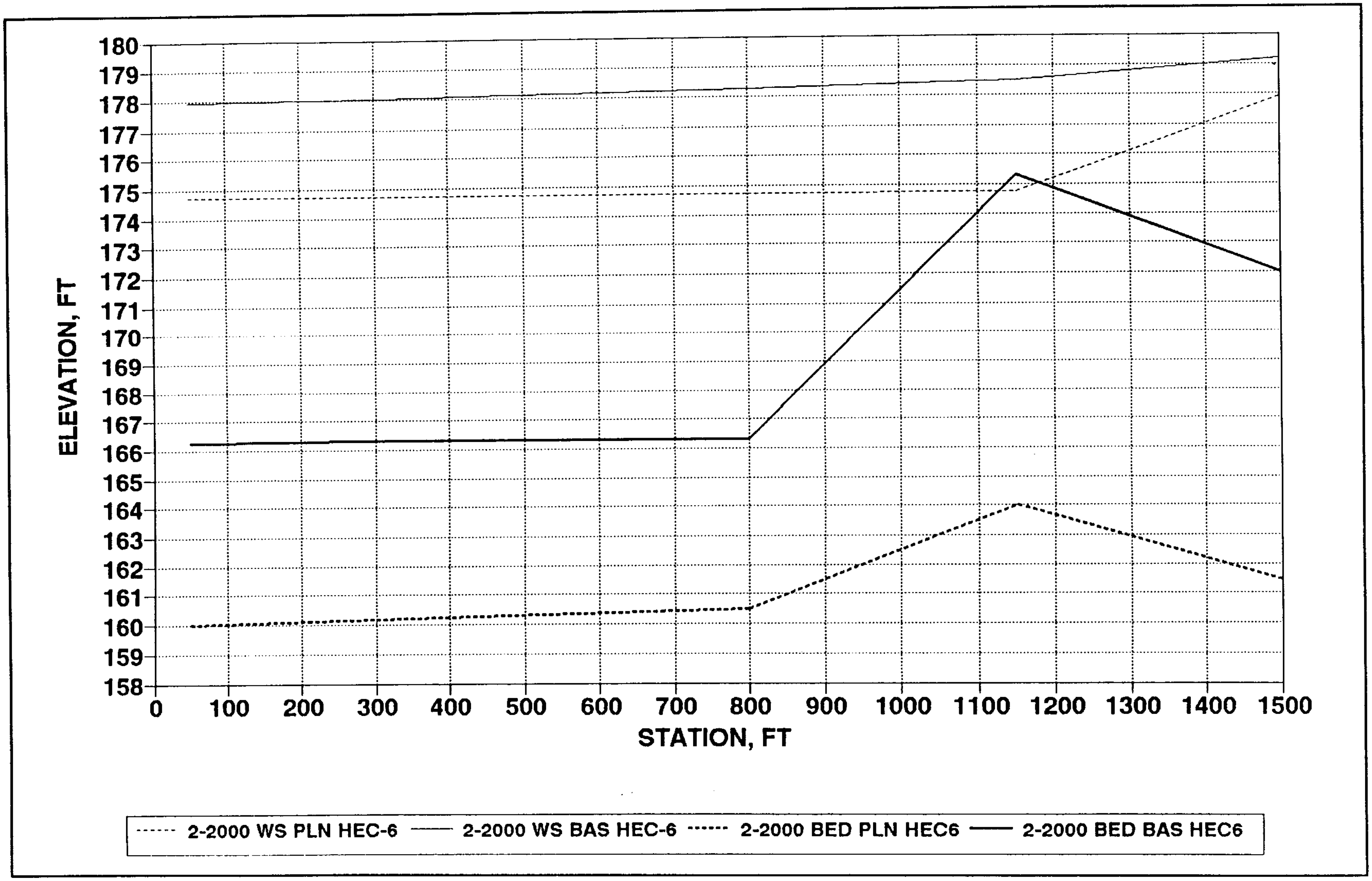


Figure E13. Base versus plan initial condition, bed and water-surface profiles, Branch 10, Bypass

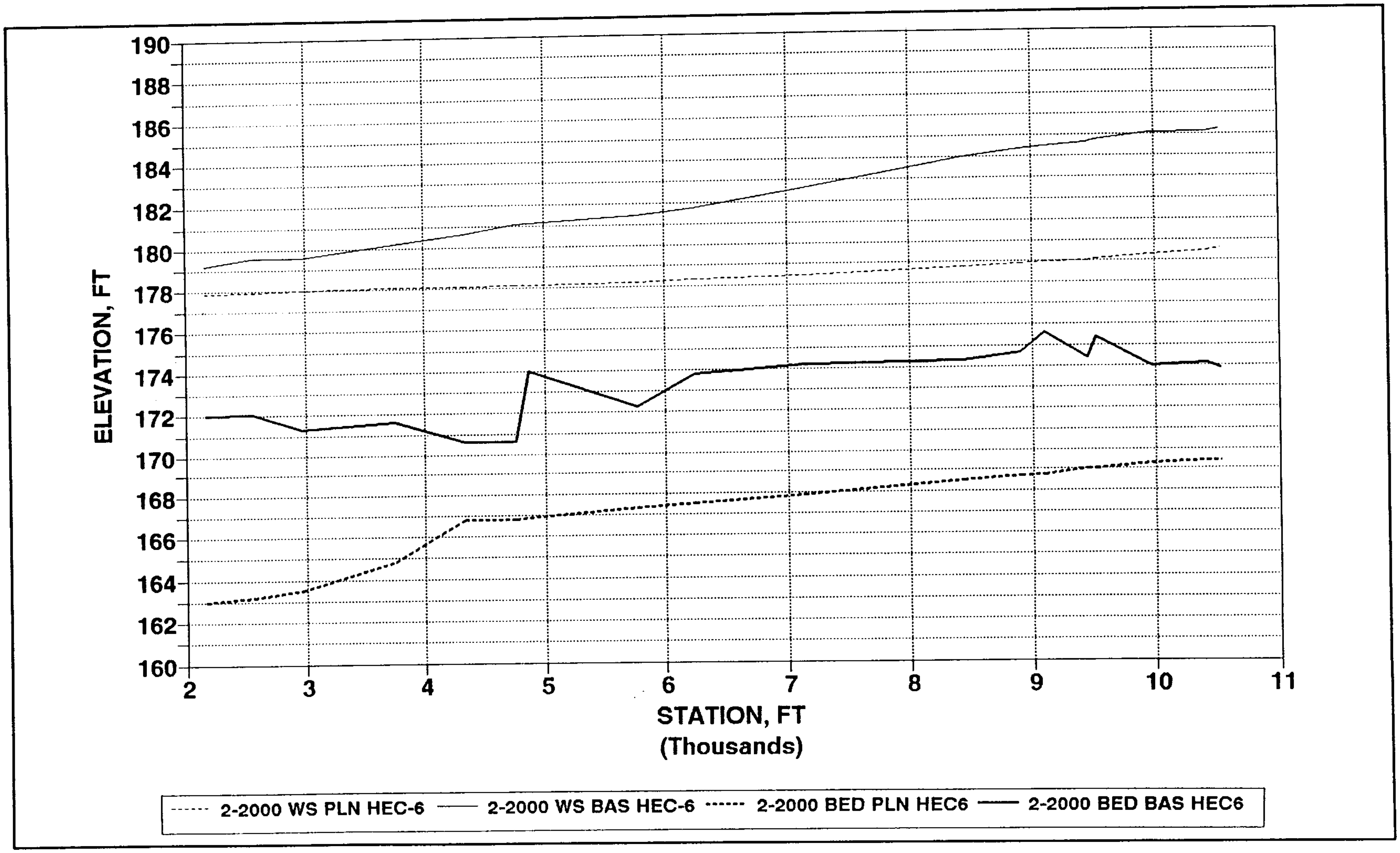


Figure E14. Base versus plan initial condition, bed and water-surface profiles, Branch 11, Pequannock River

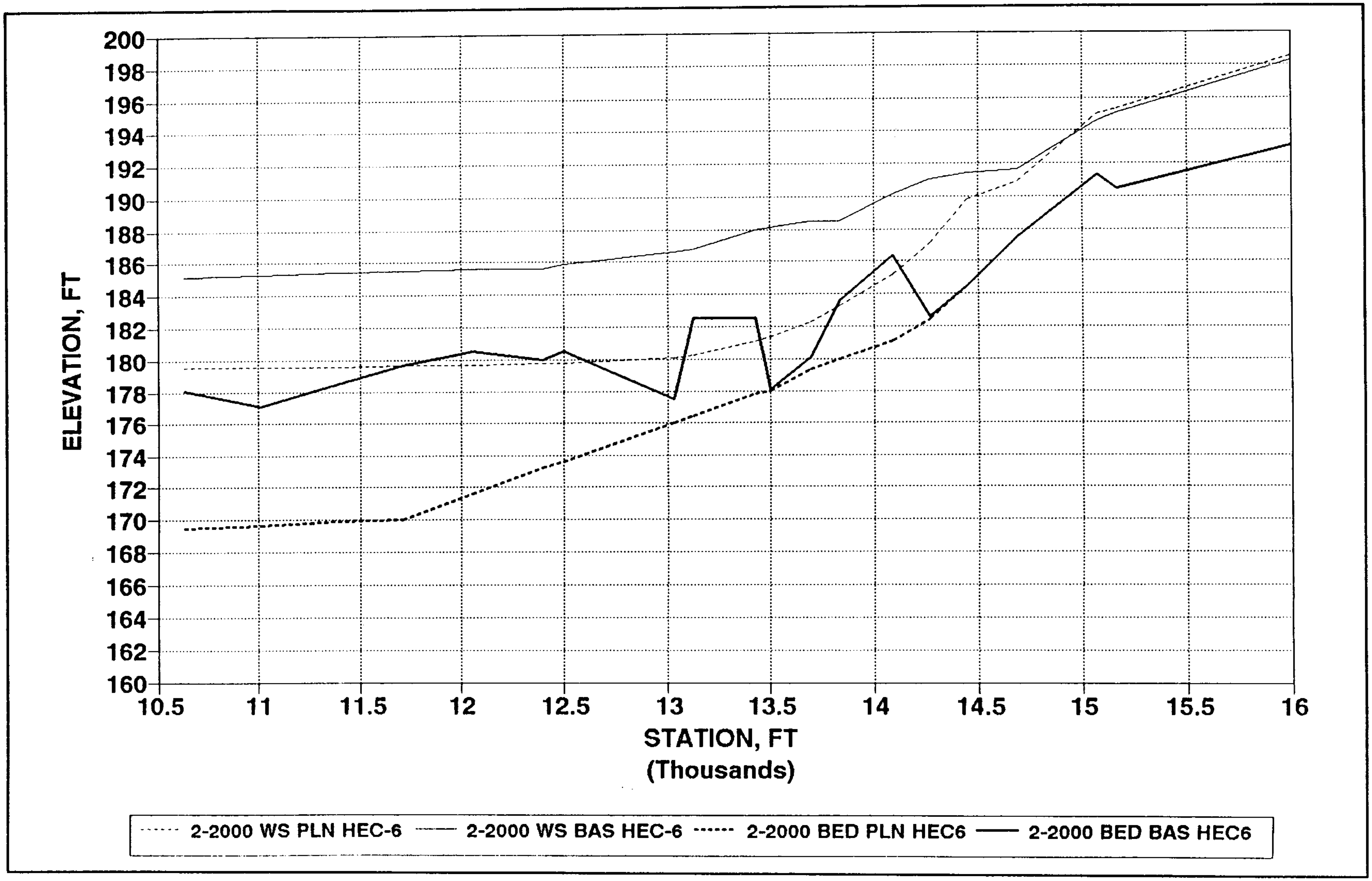


Figure E15. Base versus plan initial condition, bed and water-surface profiles, Branch 12, Pequannock River

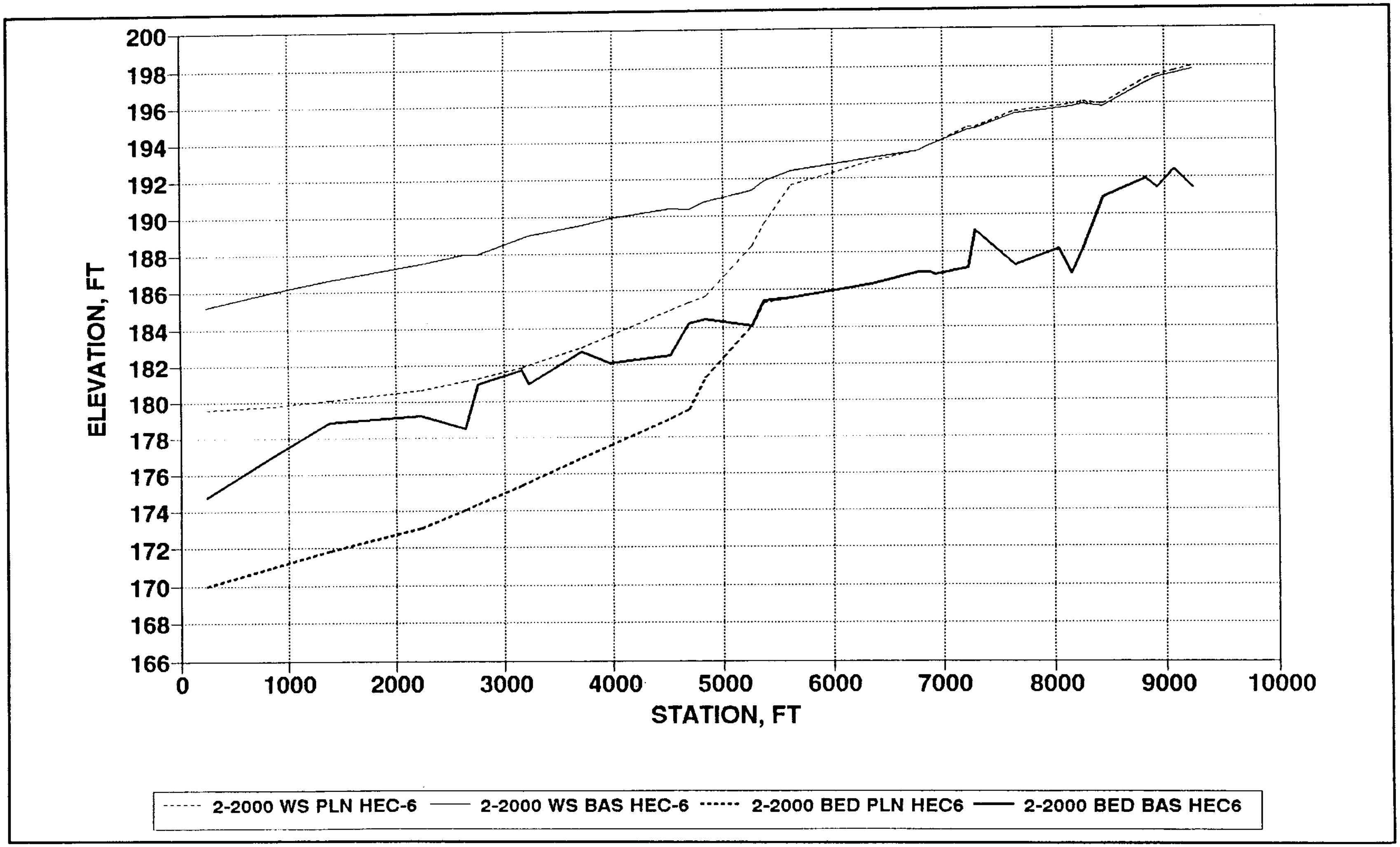


Figure E16. Base versus plan initial condition, bed and water-surface profiles, Branch 13, Wanaque River

Appendix F
Base Versus Plan Long-Term Simulation

This appendix contains graphs for river branches 1 to 13 showing the differences between base and plan beds and base and plan water surfaces after 50 years of simulation (year 2050) and using a 2-year steady state peak flow. To convert feet to meters, multiply by 0.3048.

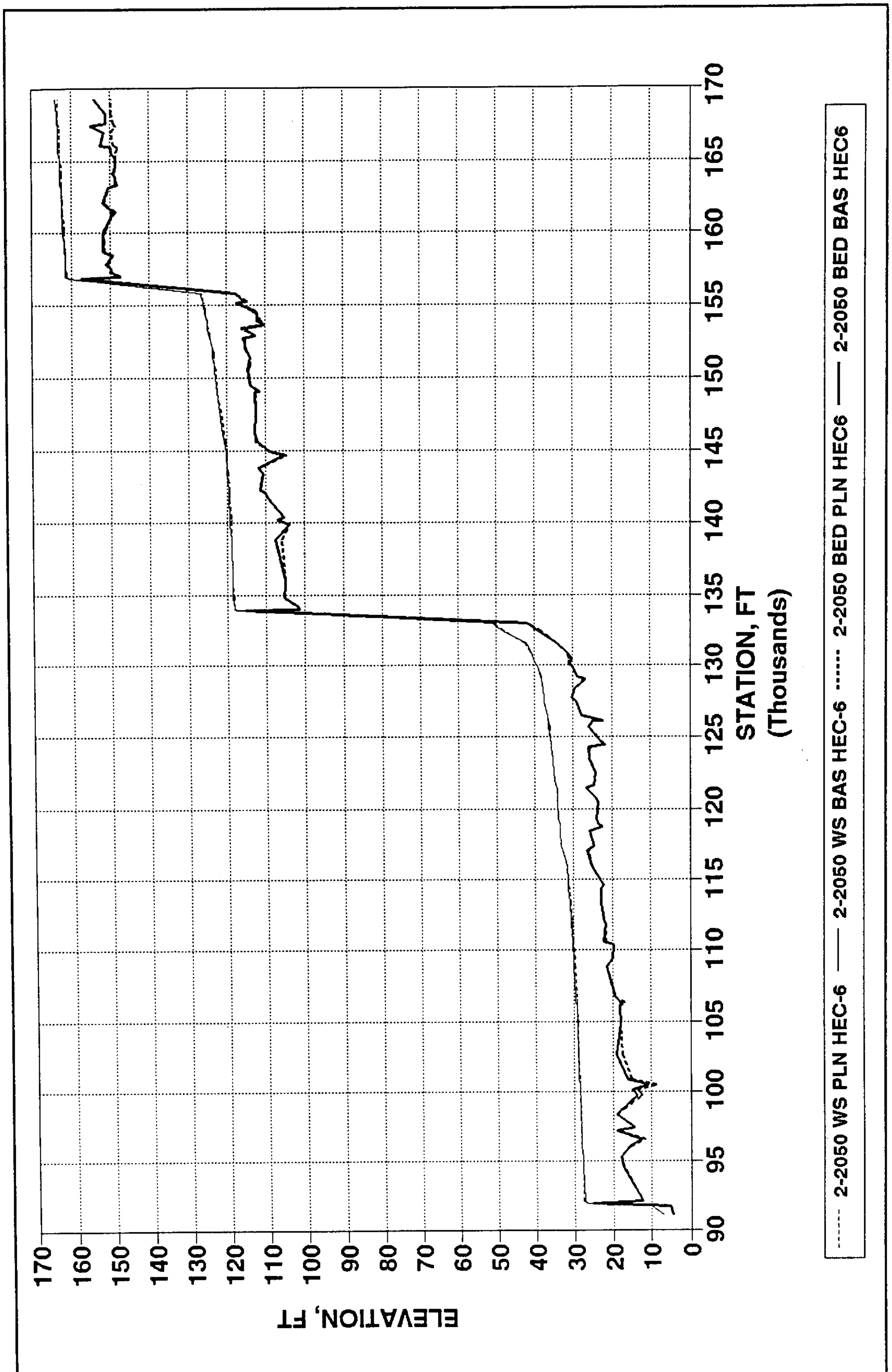


Figure F1. Base versus plan long-term simulation, bed and water-surface profiles, Branch 1, Passaic River

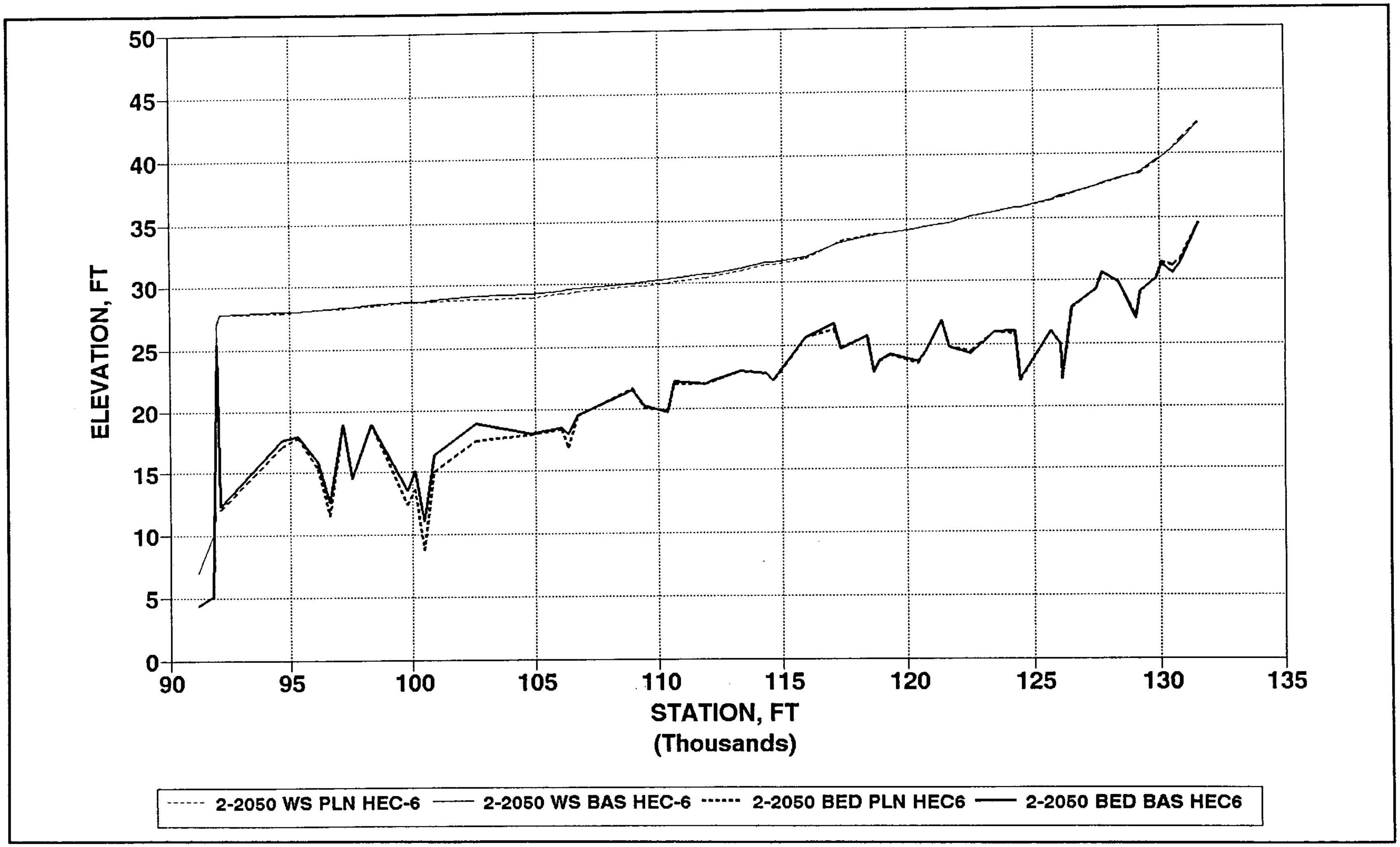


Figure F2. Base versus plan long-term simulation, bed and water-surface profiles, Branch 1, Passaic River, Dundee Dam

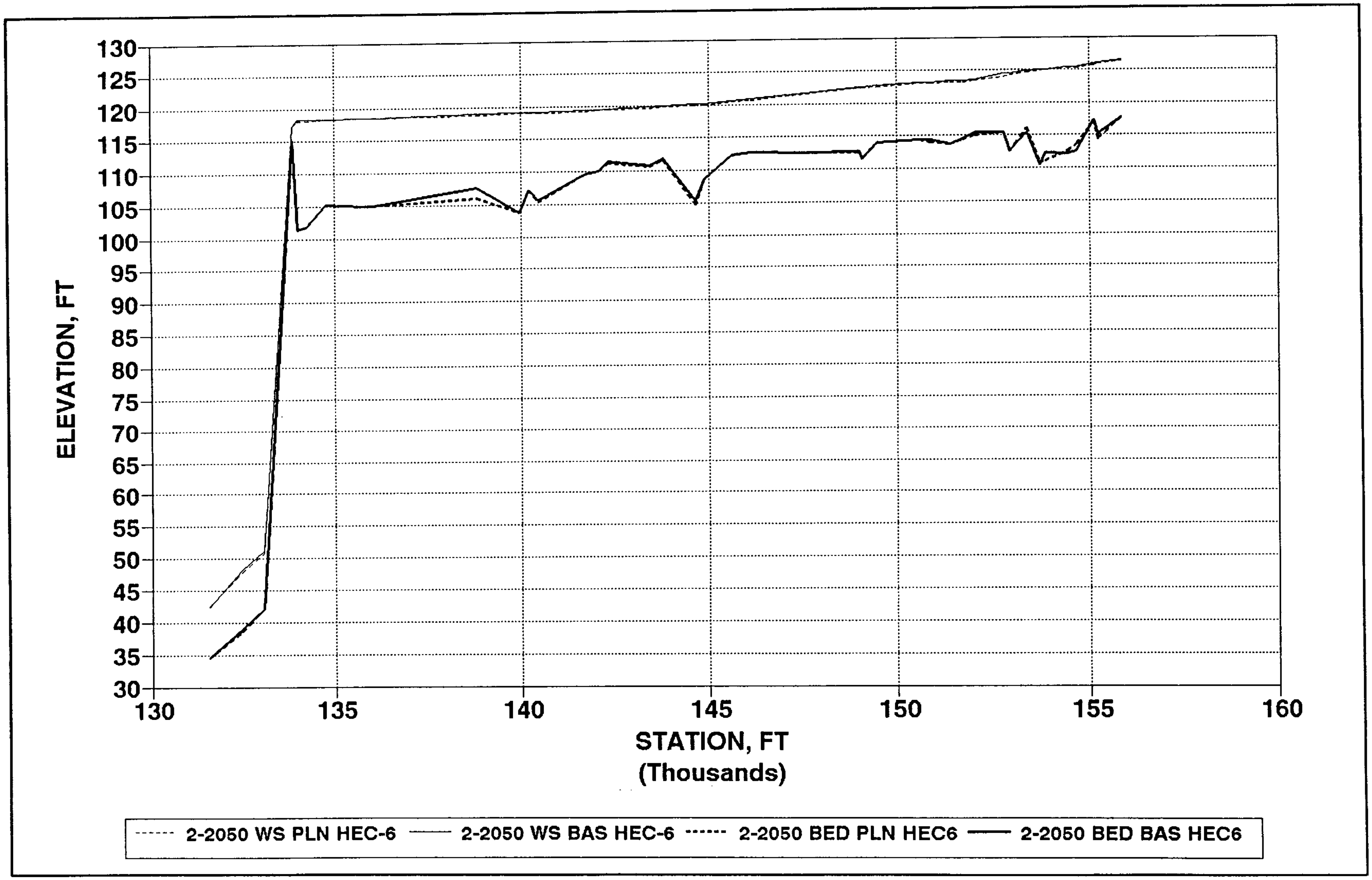


Figure F3. Base versus plan long-term simulation, bed and water-surface profiles, Branch 1, Passaic River, S.U.M. Dam

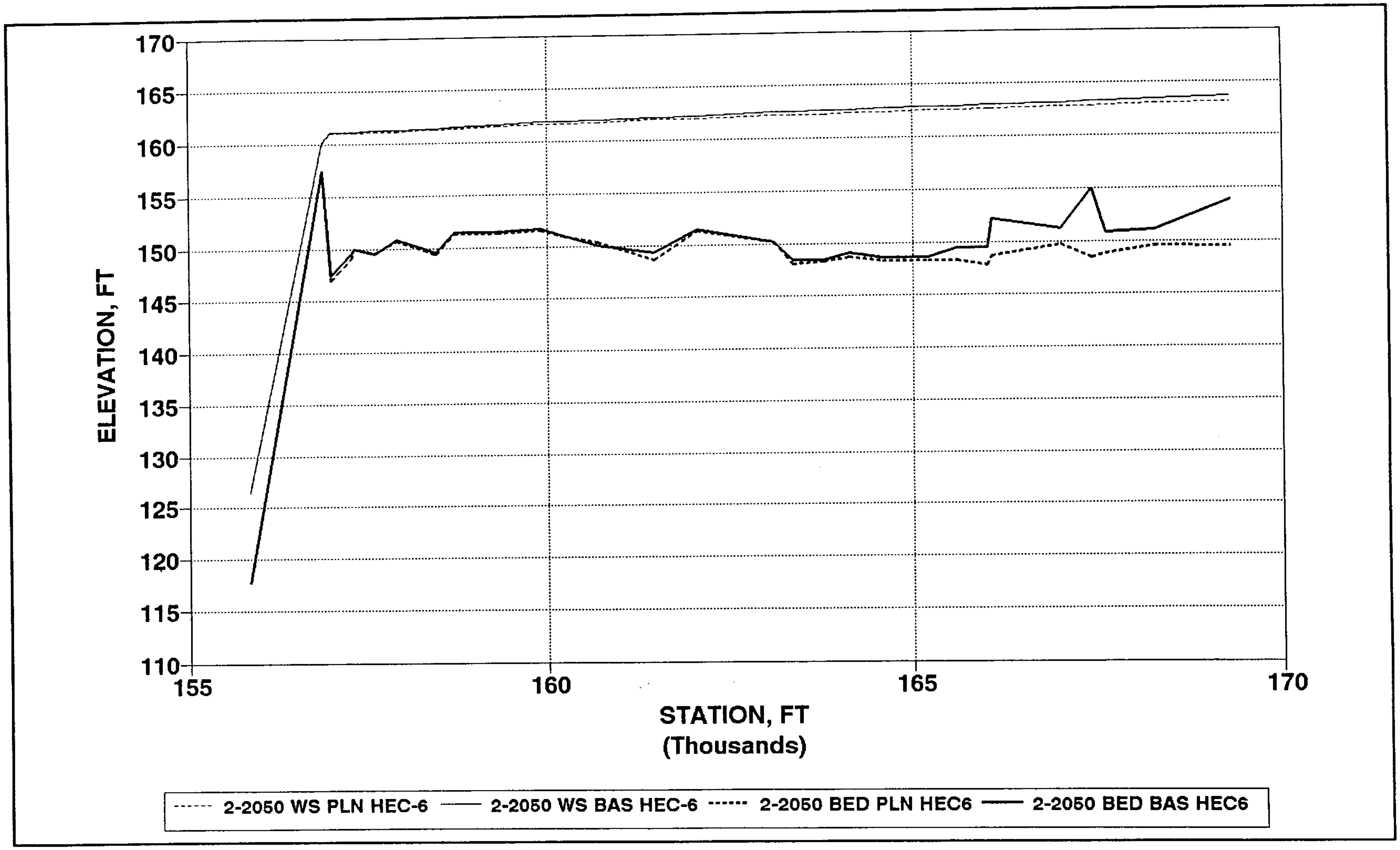


Figure F4. Base versus plan long-term simulation, bed and water-surface profiles, Branch 1, Passaic River, Beatties Dam

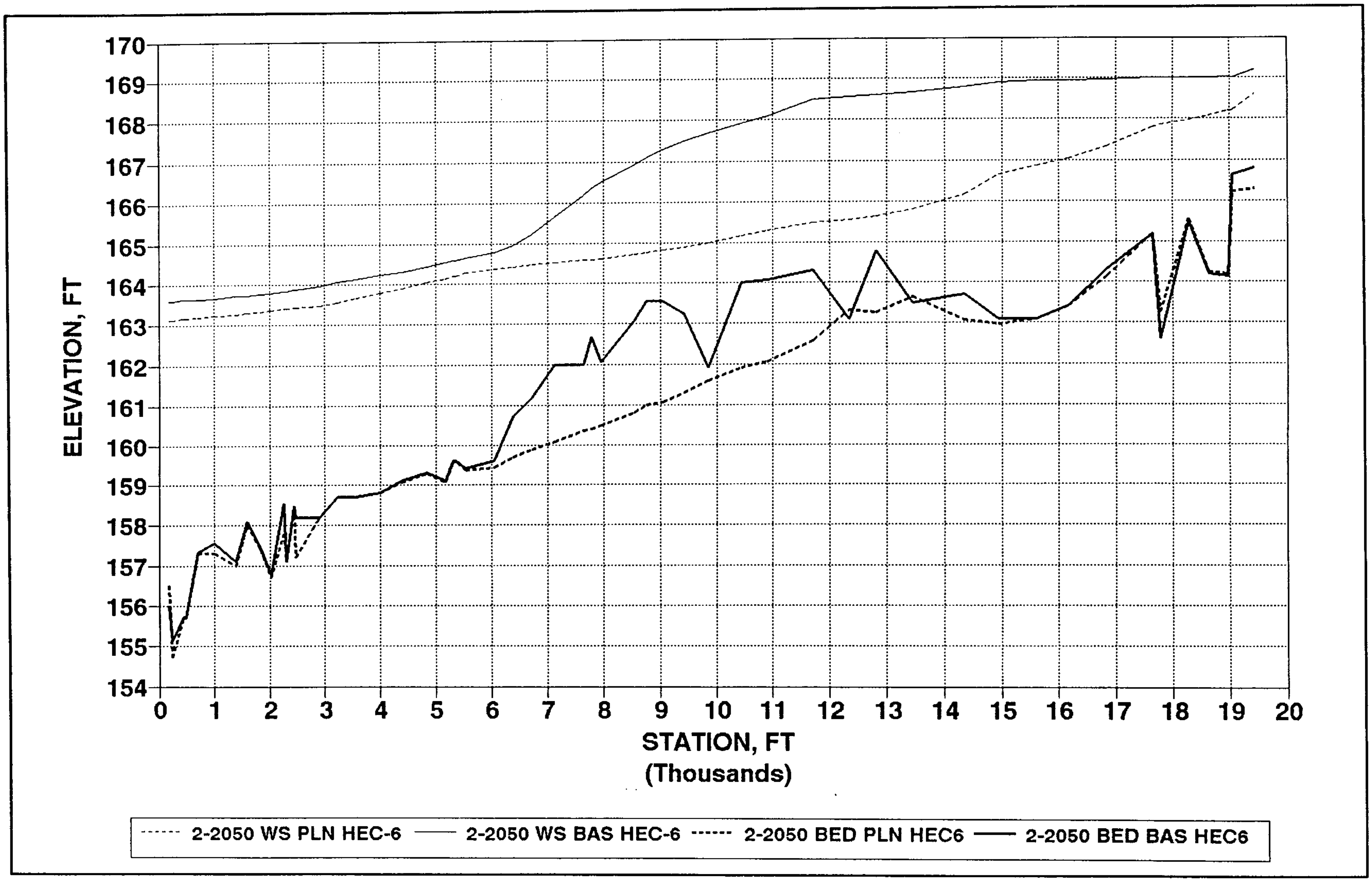


Figure F5. Base versus plan long-term simulation, bed and water-surface profiles, Branch 2, Deepavaal Brook

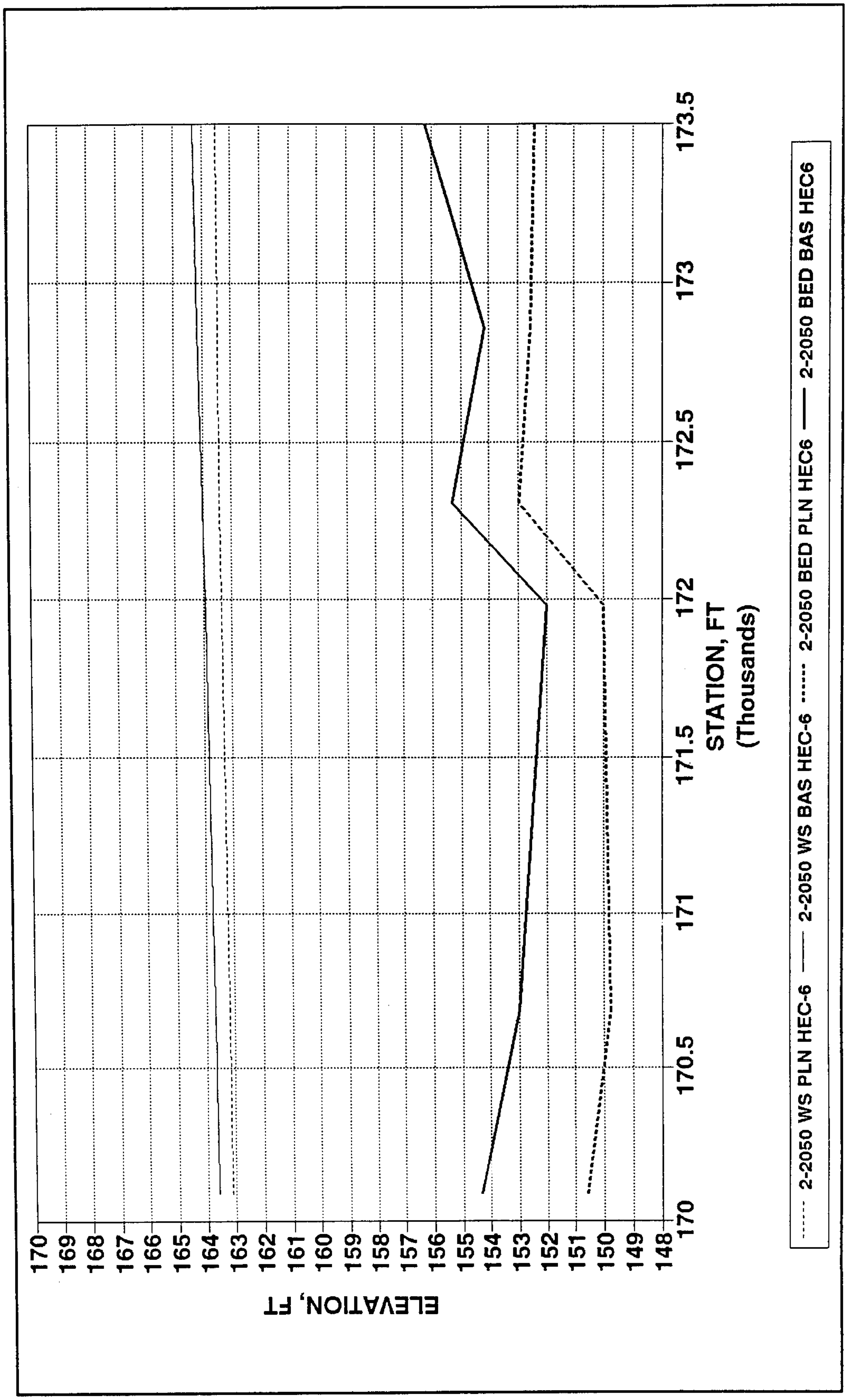


Figure F6. Base versus plan long-term simulation, bed and water-surface profiles, Branch 3, Passaic River

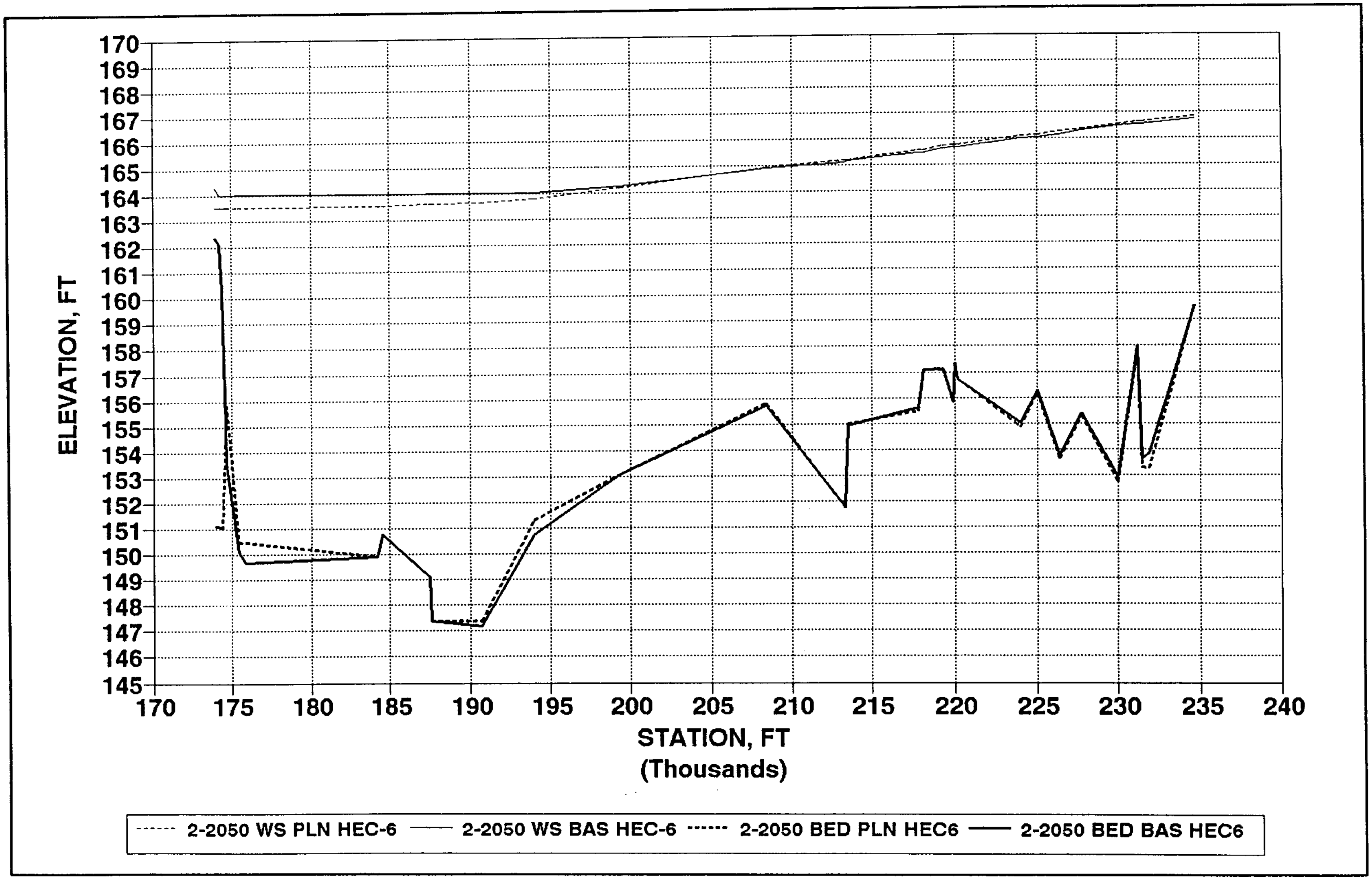


Figure F7. Base versus plan long-term simulation, bed and water-surface profiles, Branch 4, Passaic River

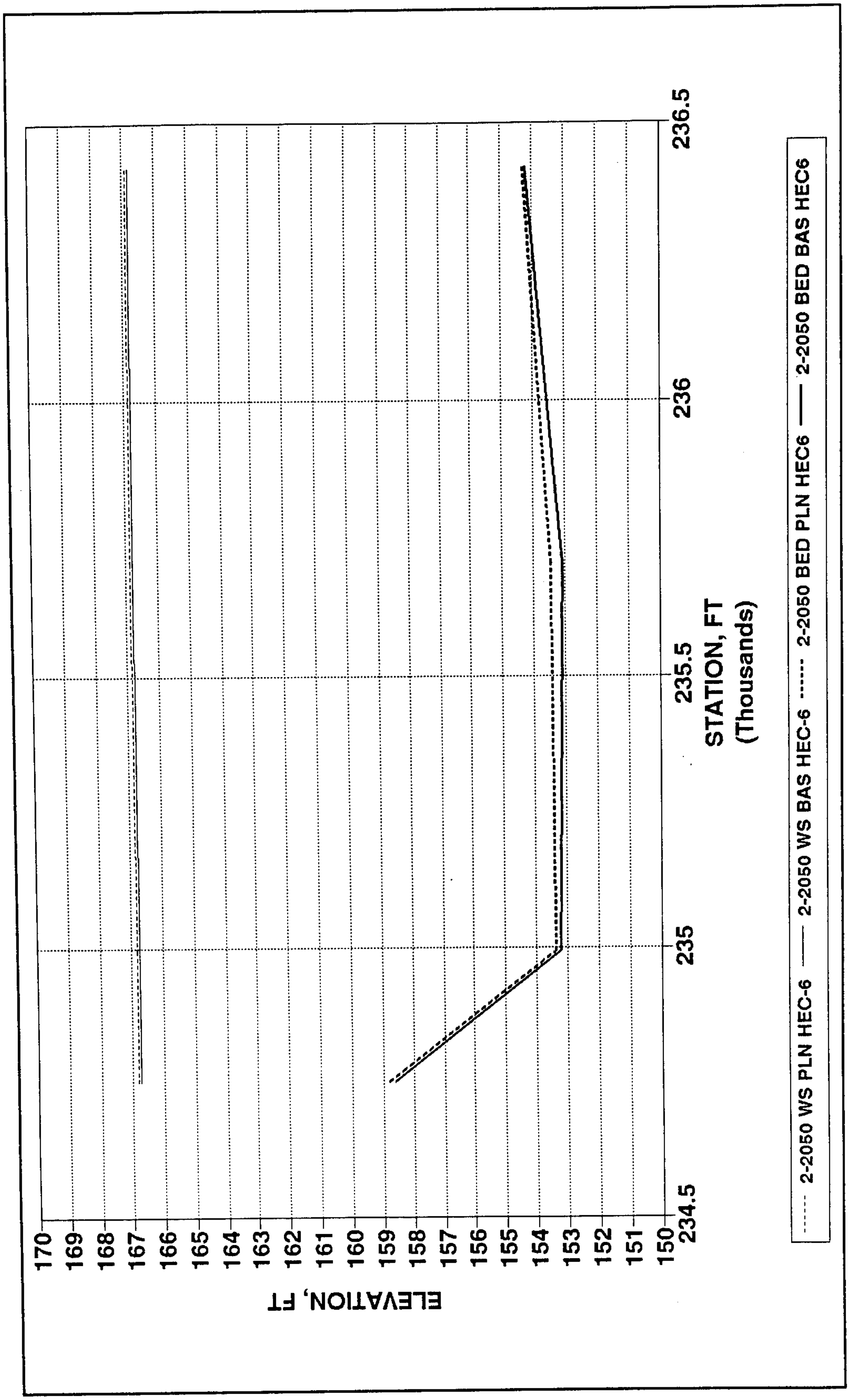


Figure F8. Base versus plan long-term simulation, bed and water-surface profiles, Branch 5, Passaic River

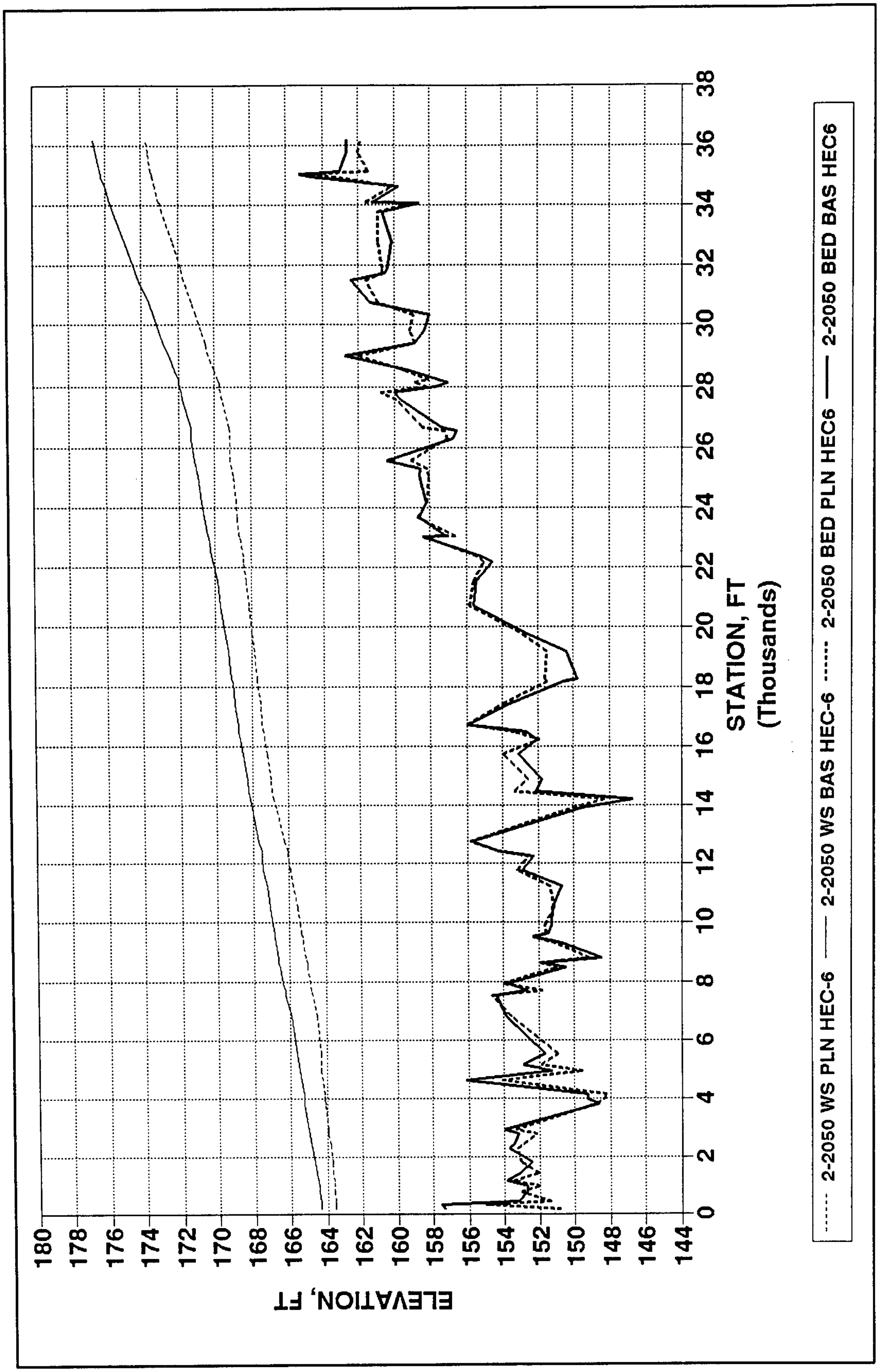


Figure F9. Base versus plan long-term simulation, bed and water-surface profiles, Branch 6, Pompton River

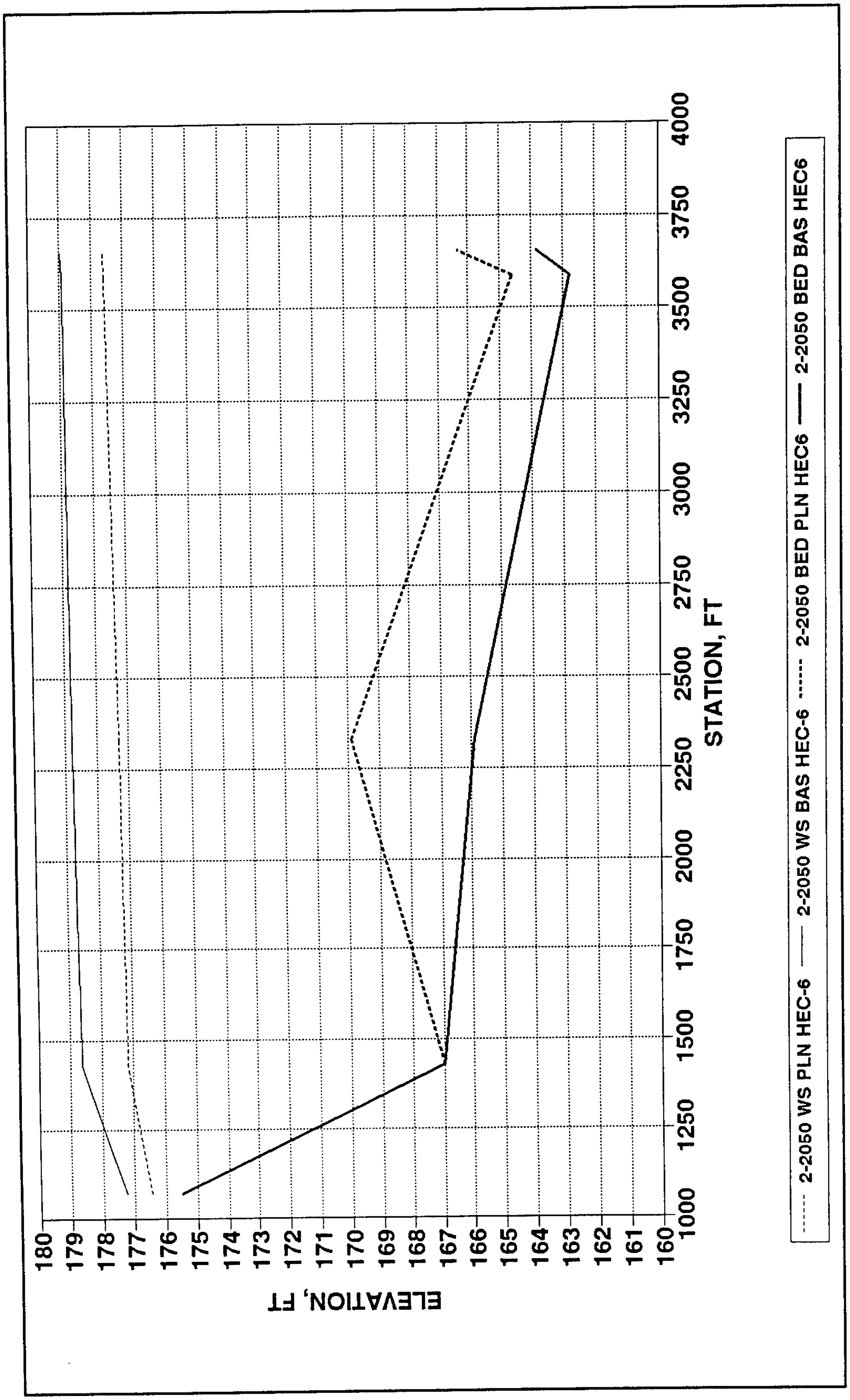


Figure F10. Base versus plan long-term simulation, bed and water-surface profiles, Branch 7, Ramapo River

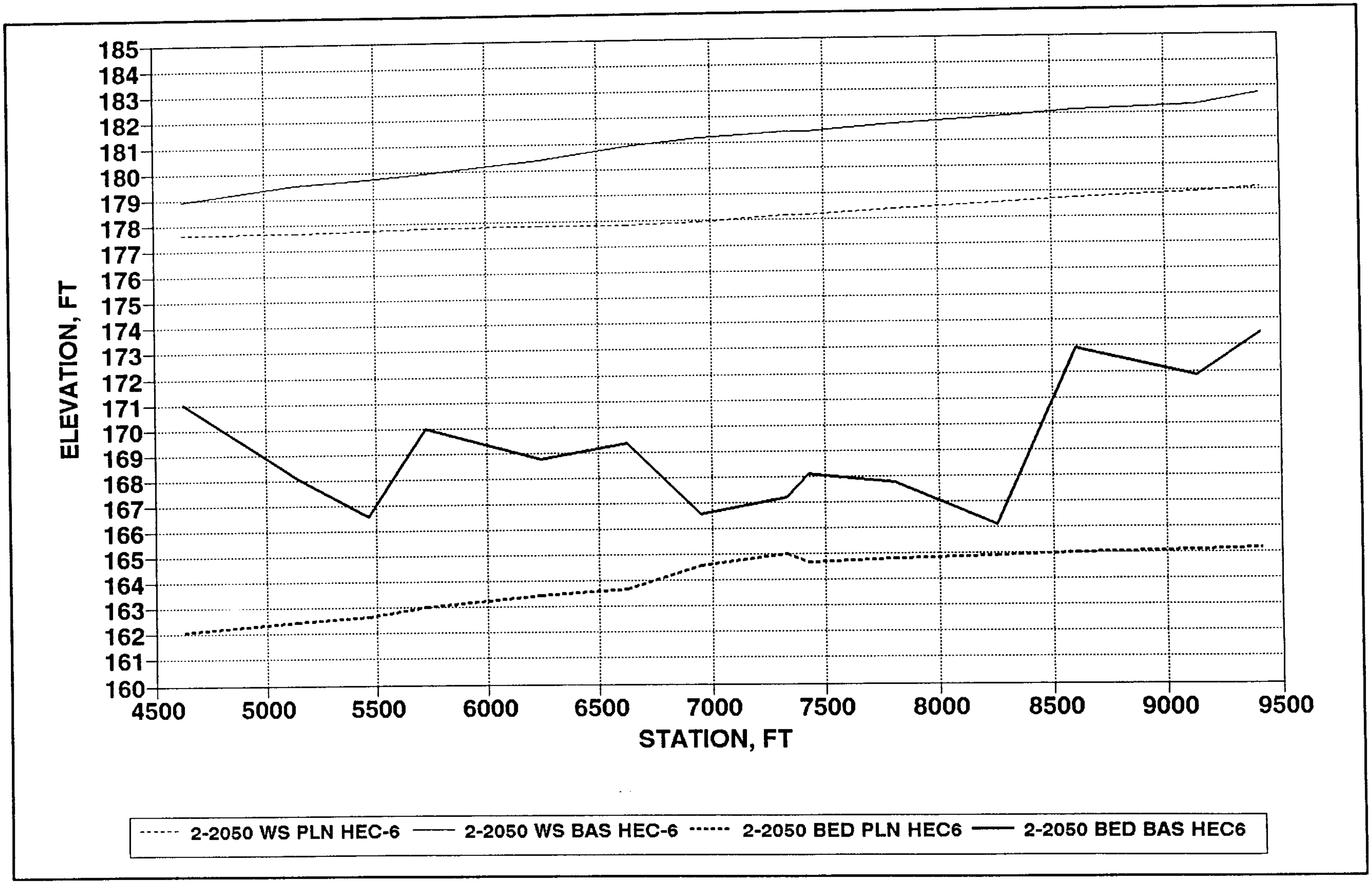


Figure F11. Base versus plan long-term simulation, bed and water-surface profiles, Branch 8, Ramapo River

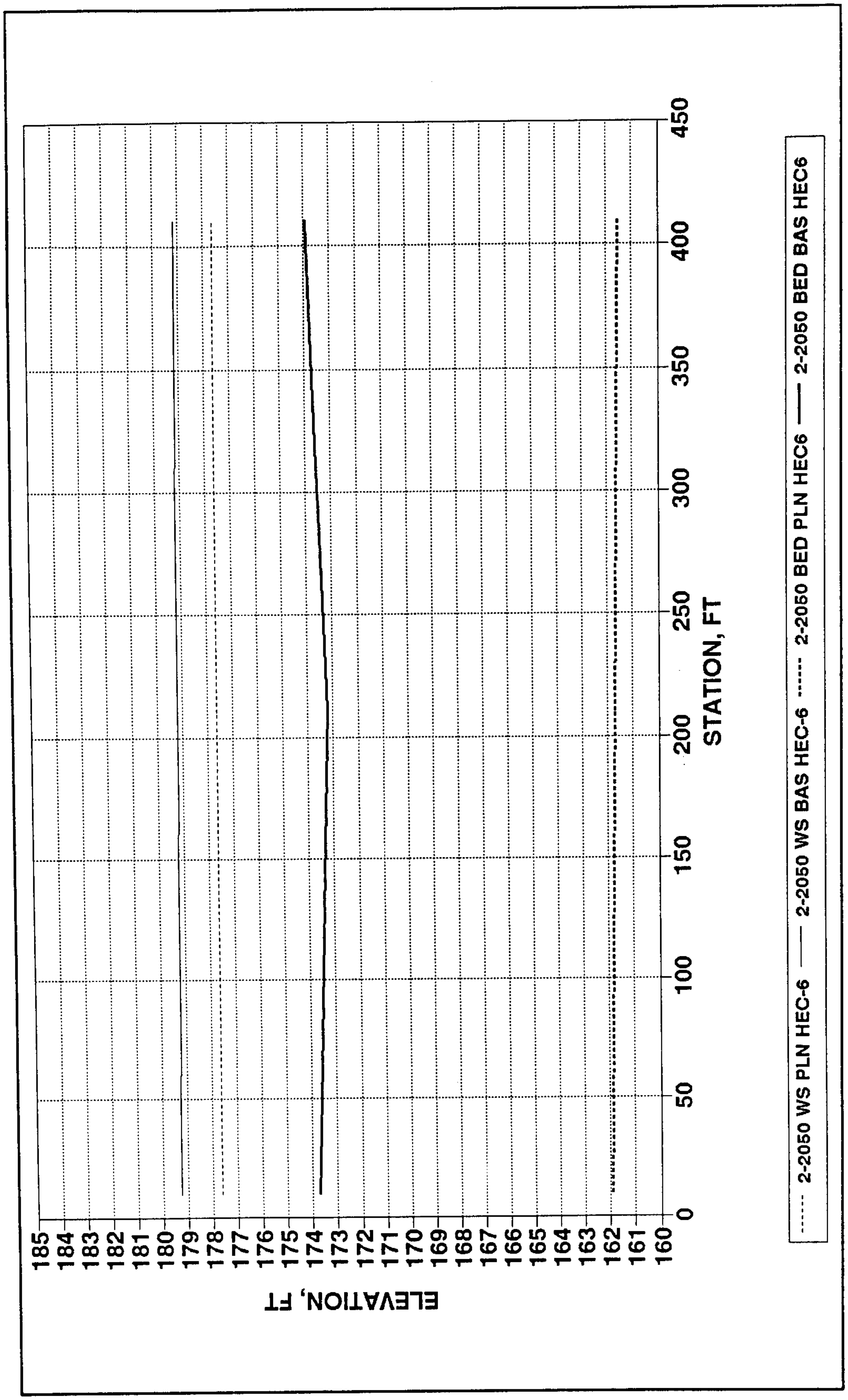


Figure F12. Base versus plan long-term simulation, bed and water-surface profiles, Branch 9

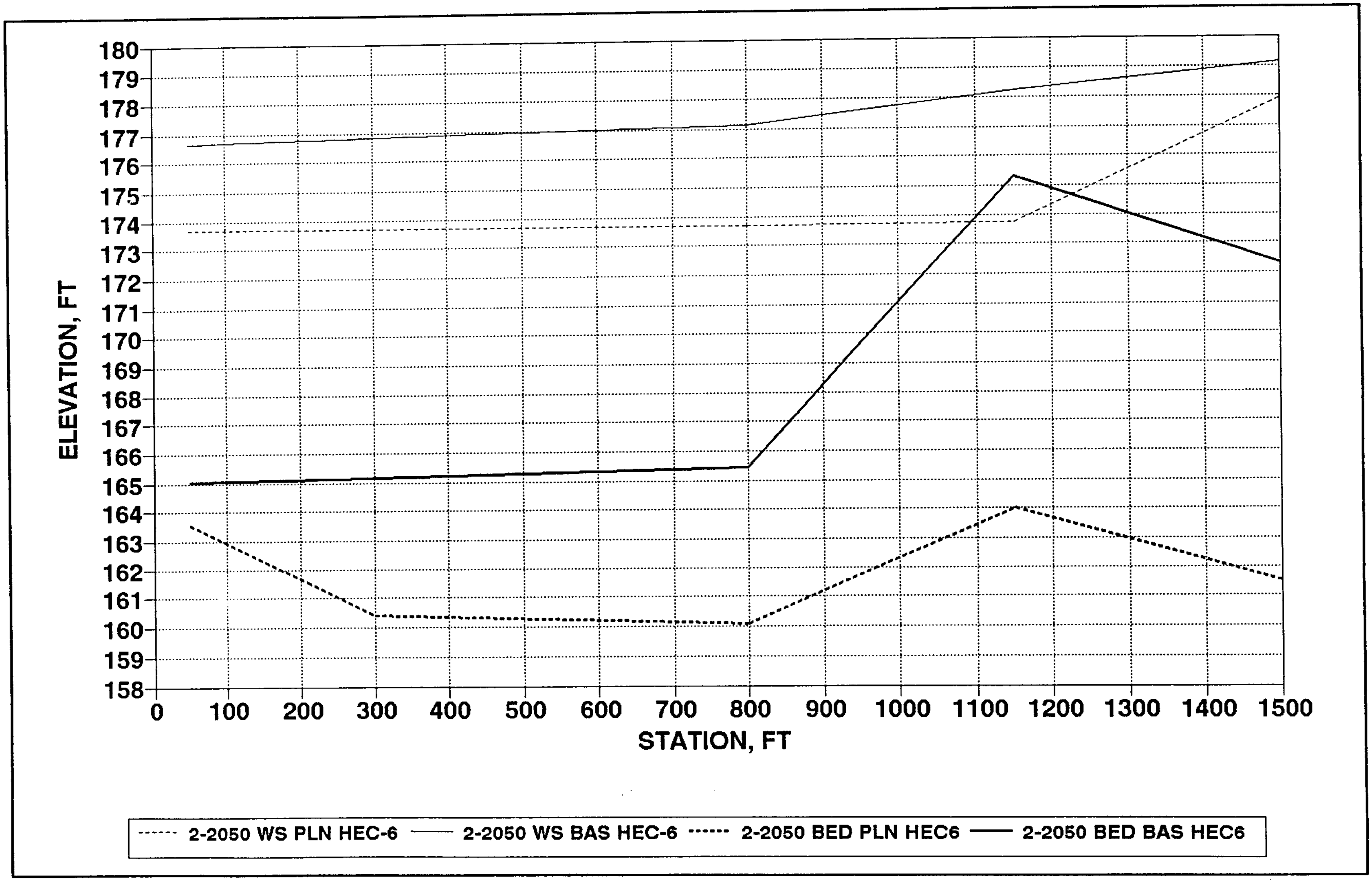


Figure F13. Base versus plan long-term simulation, bed and water-surface profiles, Branch 10, Bypass

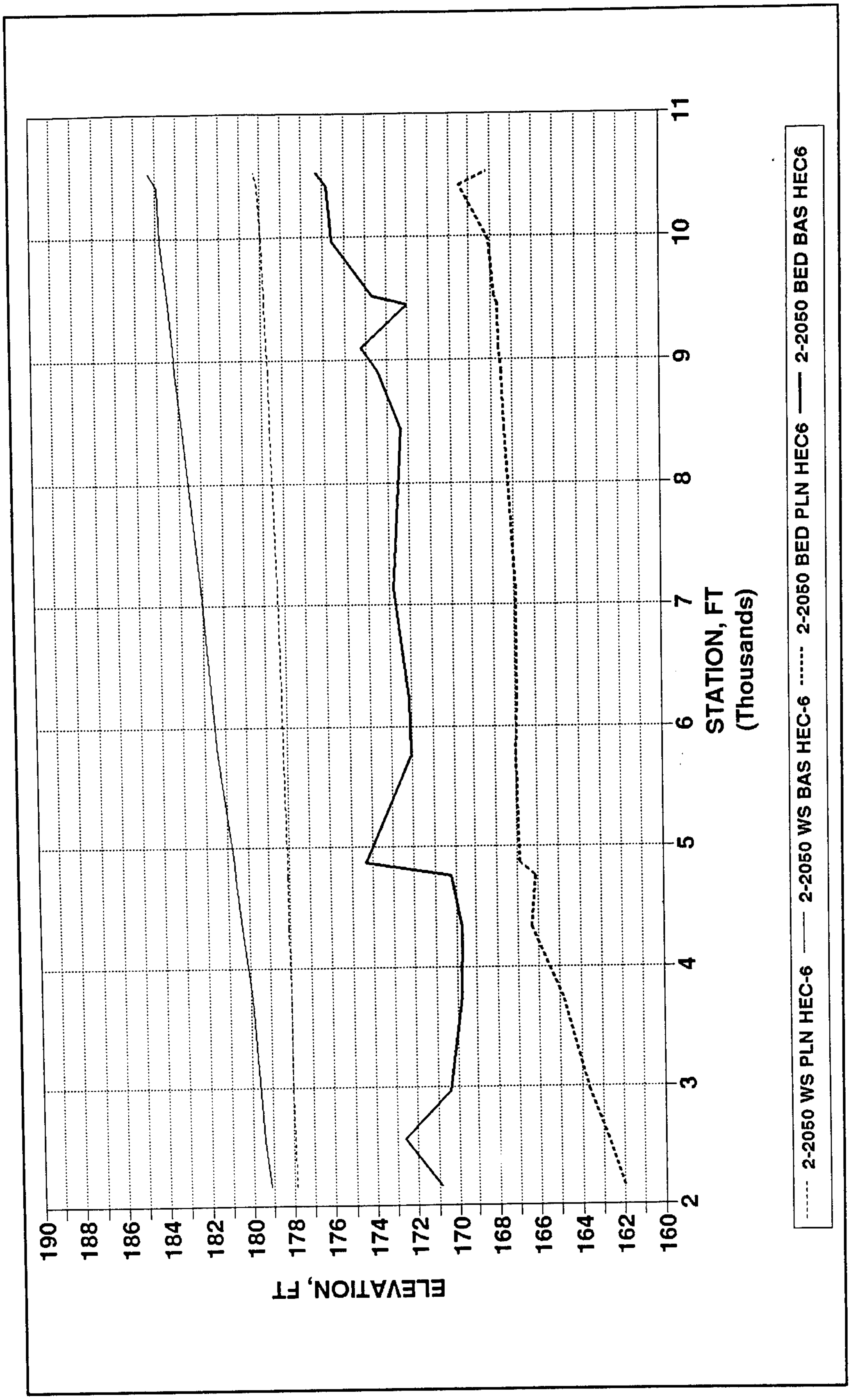


Figure F14. Base versus plan long-term simulation, bed and water-surface profiles, Branch 11, Pequannock River

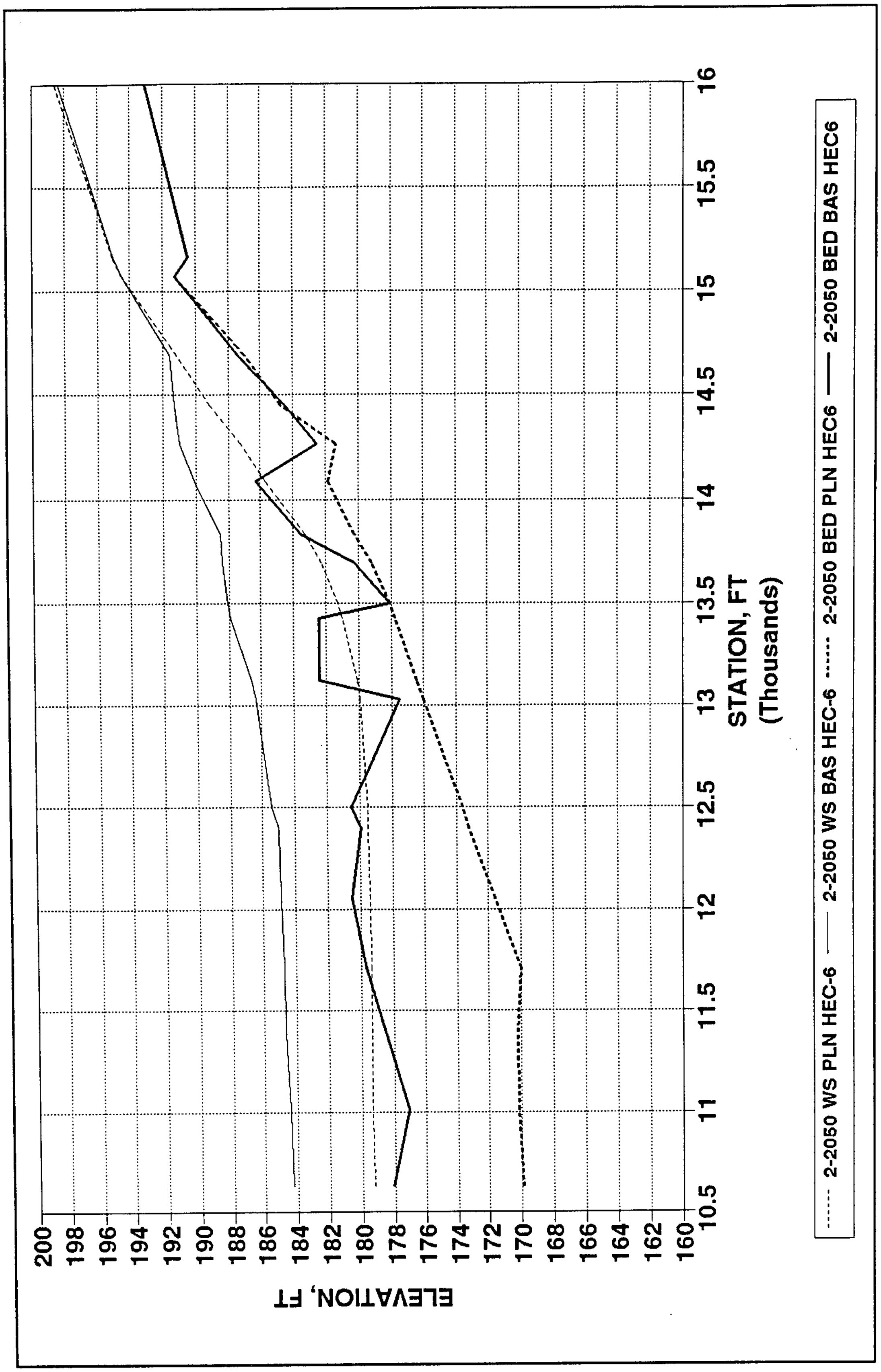


Figure F15. Base versus plan long-term simulation, bed and water-surface profiles, Branch 12, Pequannock River

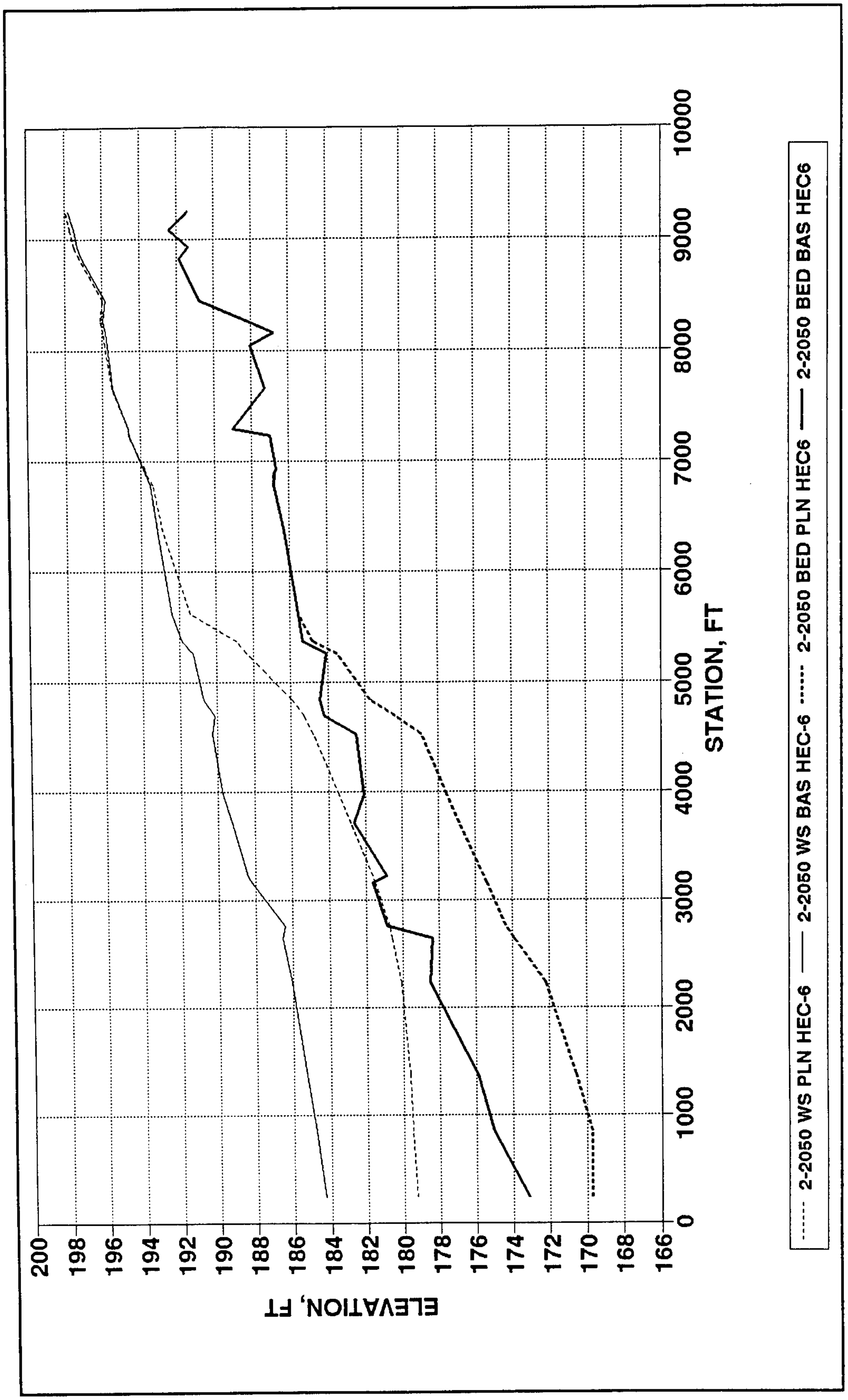


Figure F16. Base versus plan long-term simulation, bed and water-surface profiles, Branch 13, Wanaque River

Appendix G
Base and Plan Long-Term Simulation with 2-year Hydrograph

This appendix contains graphs for river branches 1 to 13 showing the differences between base and plan beds and base and plan maximum water surfaces, with a 14-day-duration 2-year hydrograph superimposed on the 50-year simulation (year 2050).

It also shows differences between plan beds and plan maximum water surfaces with a 14-day-duration 2-year hydrograph superimposed on the year 2000 and year 2050 simulations. To convert feet to meters, multiply by 0.3048.

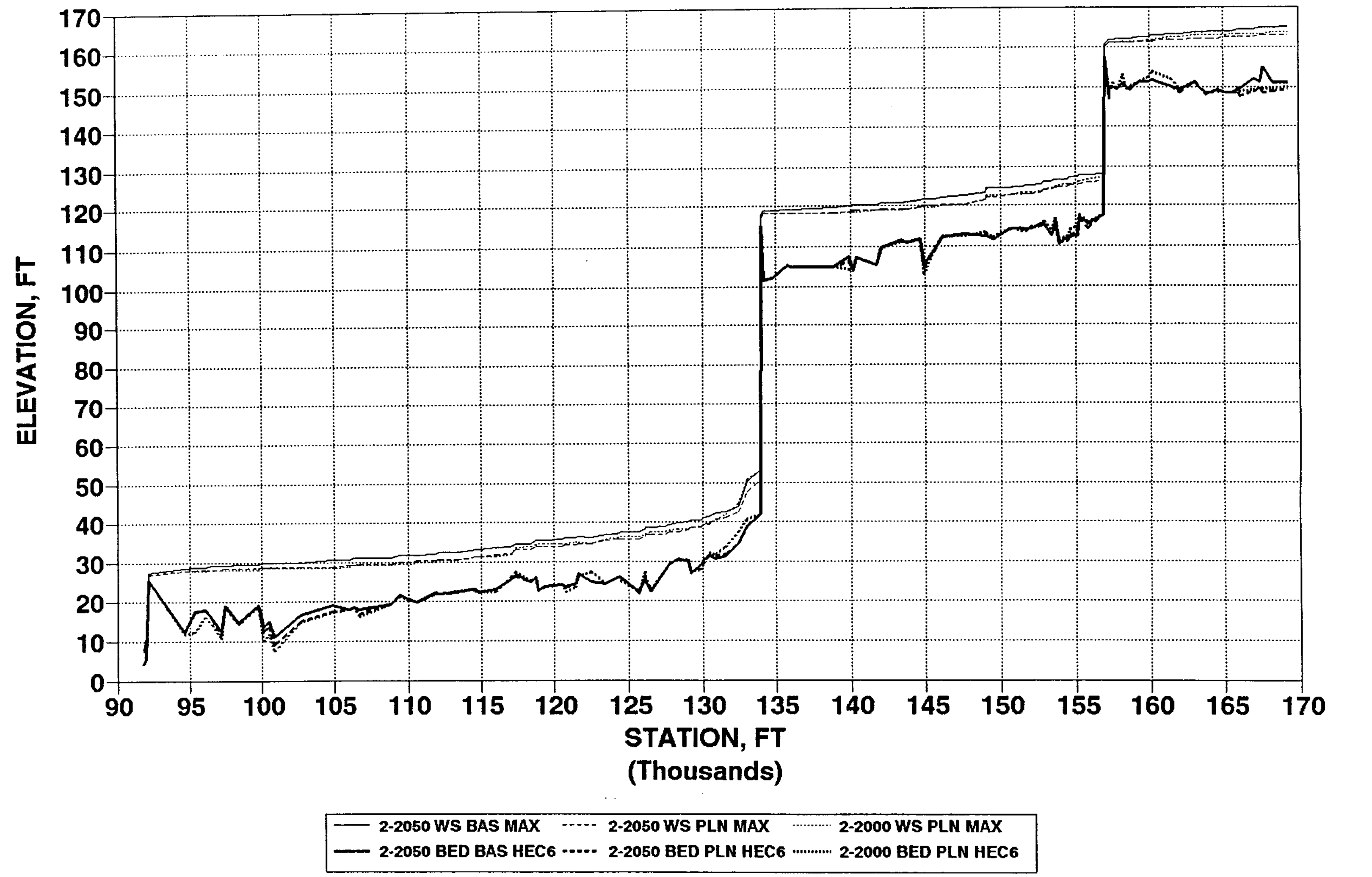
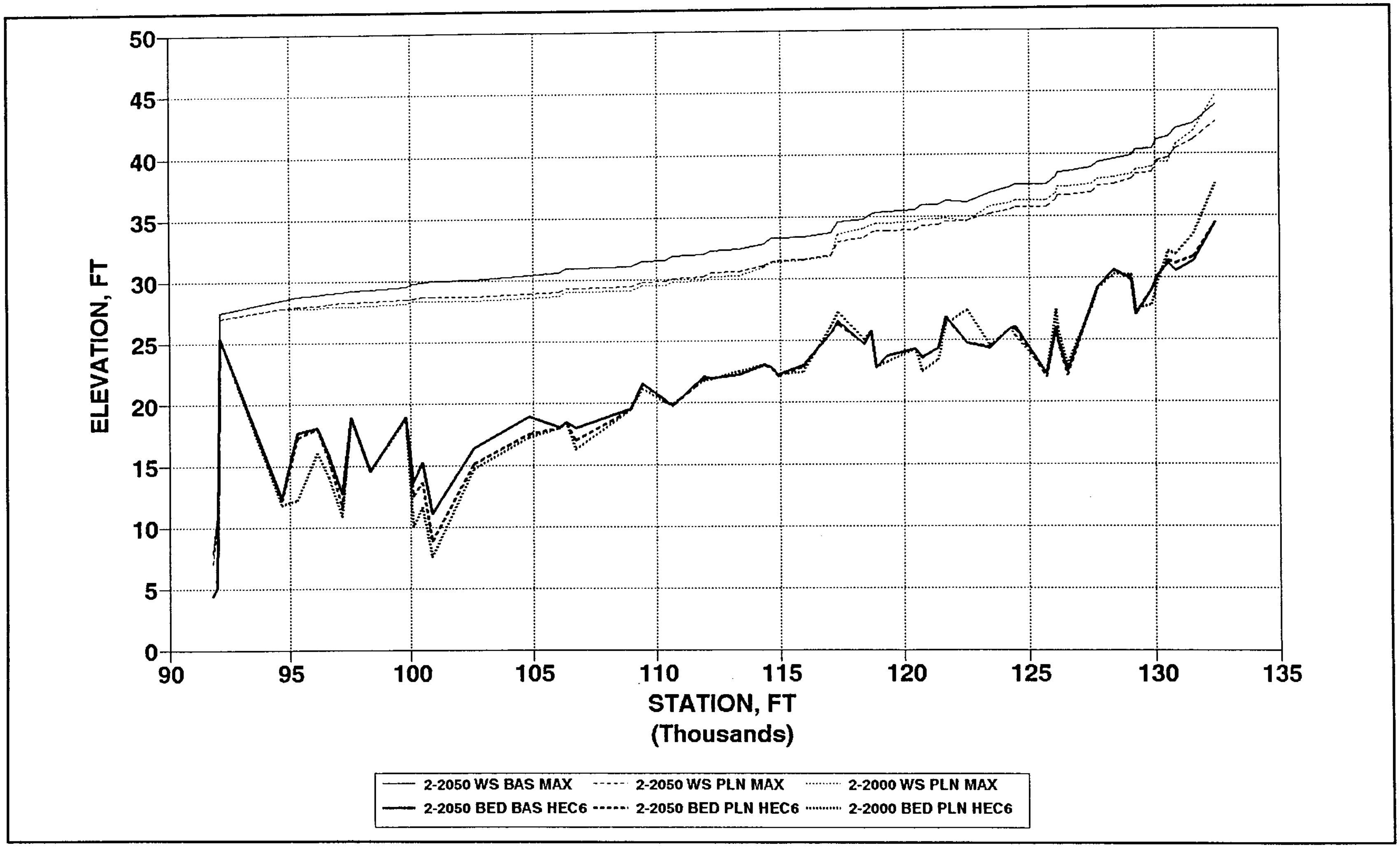


Figure G1. Base and plan long-term simulation with 2-year hydrograph, bed and water-surface profiles, Branch 1, Passaic River



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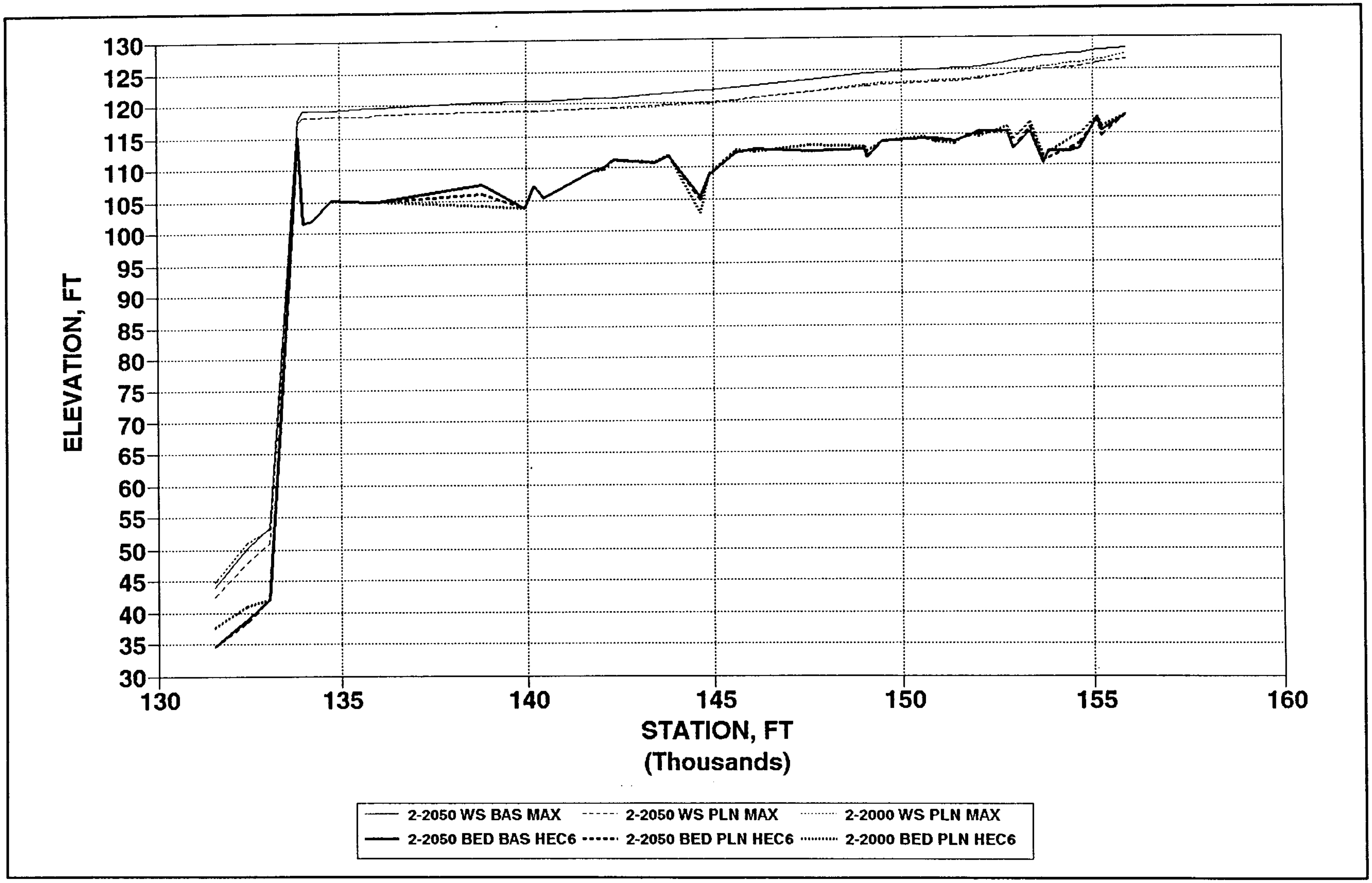
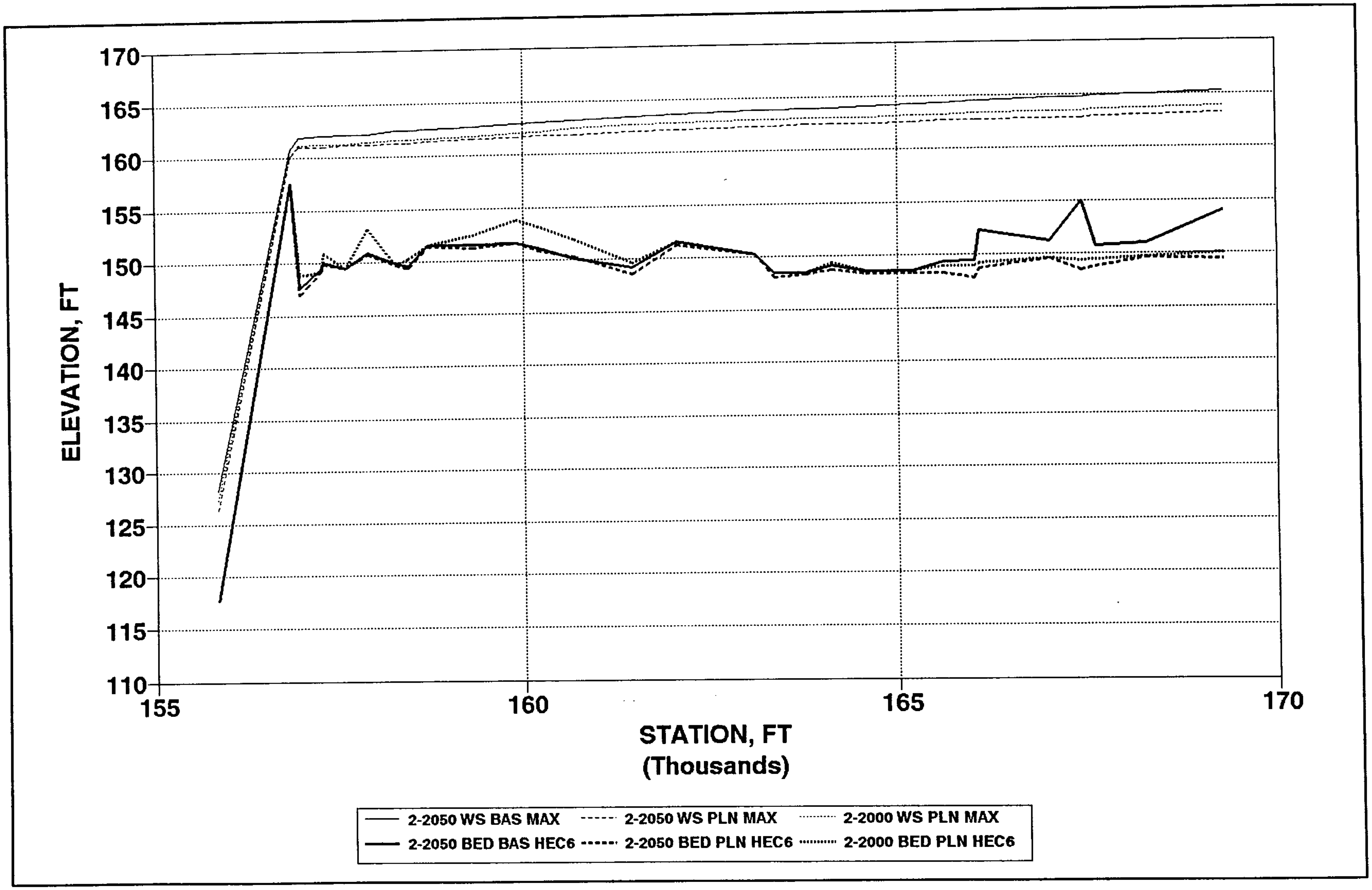


Figure G3. Base and plan long-term simulation with 2-year hydrograph, bed and water-surface profiles, Branch 1, Passaic River, S.U.M. Dam



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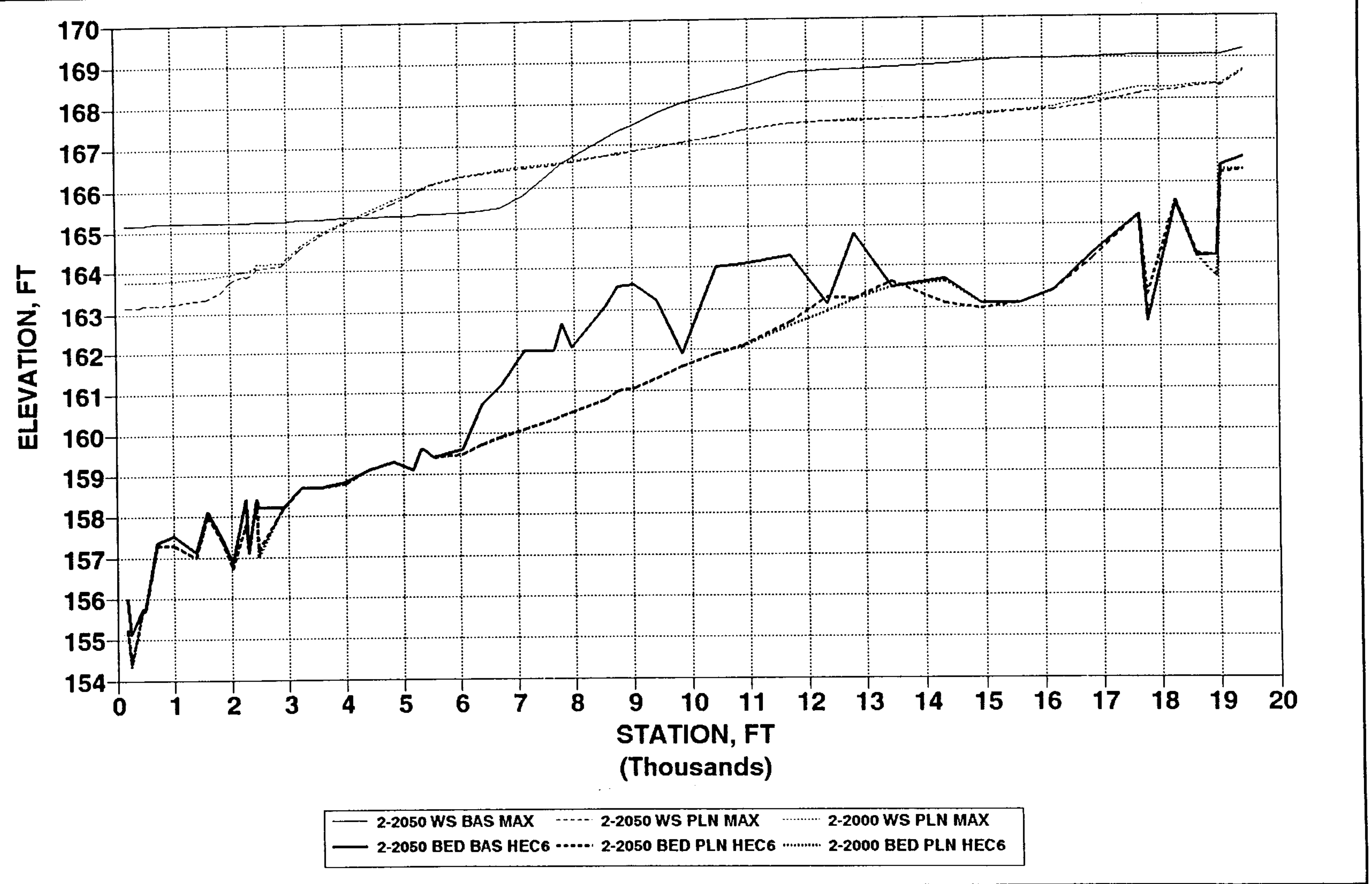

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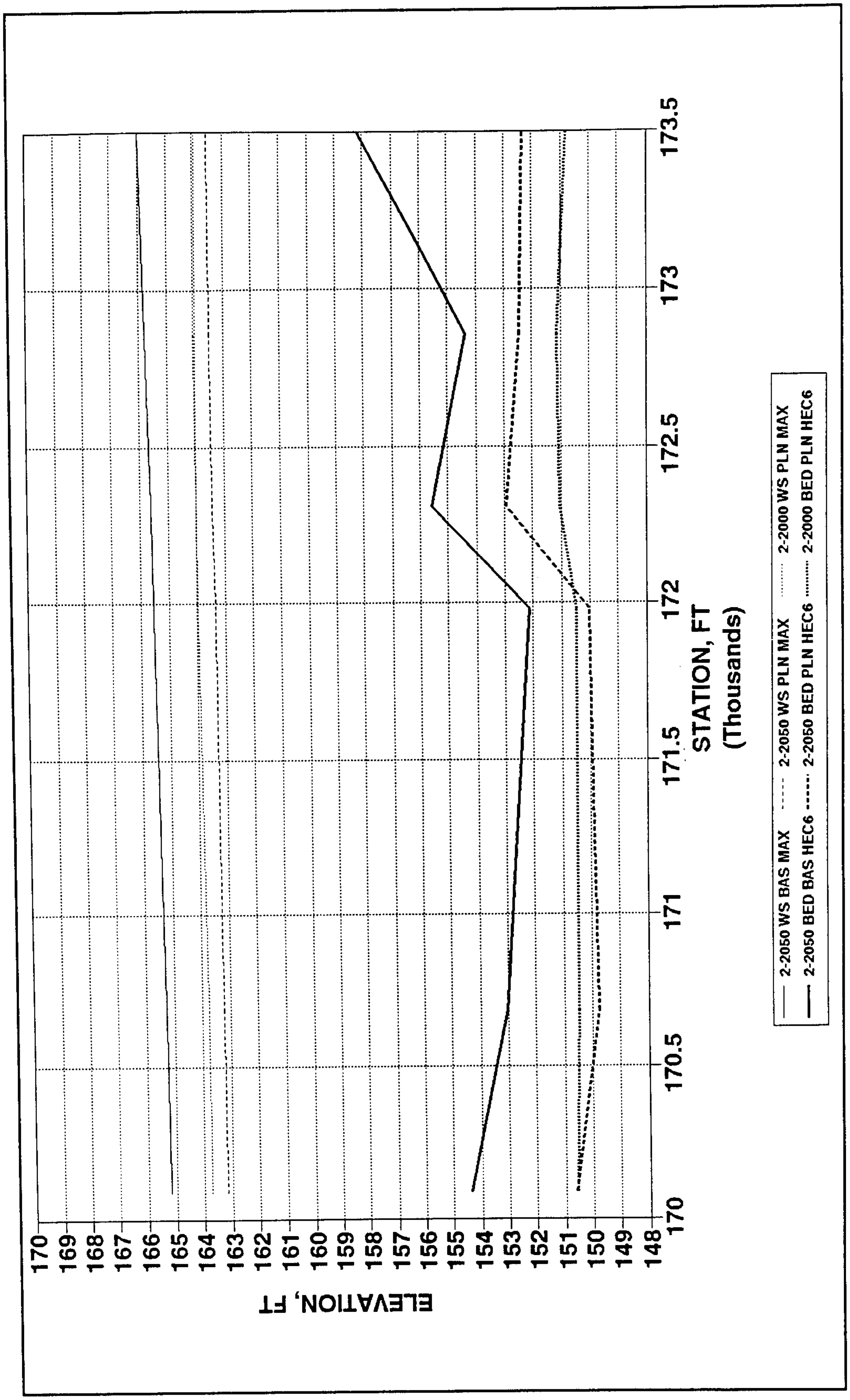

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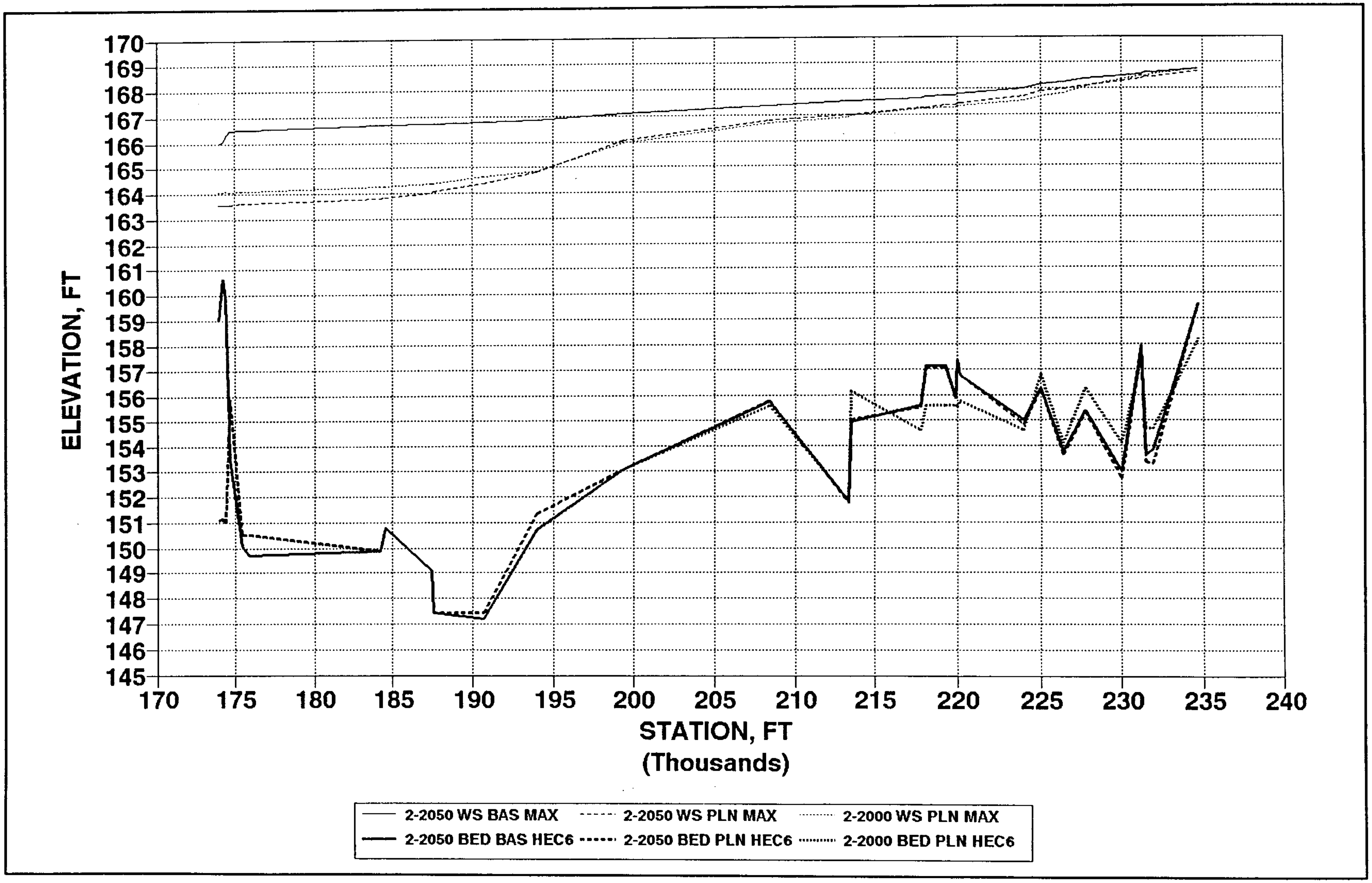

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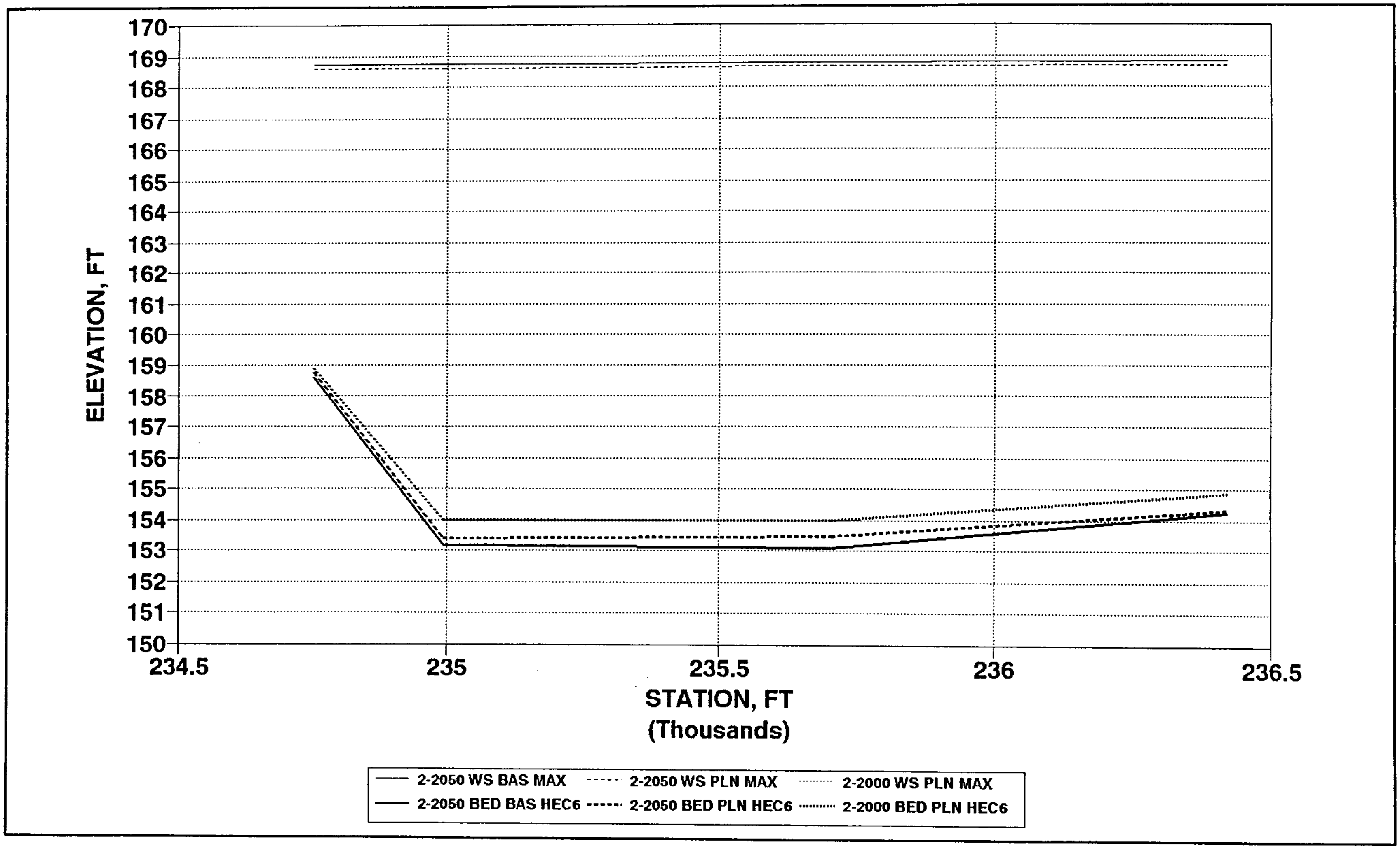

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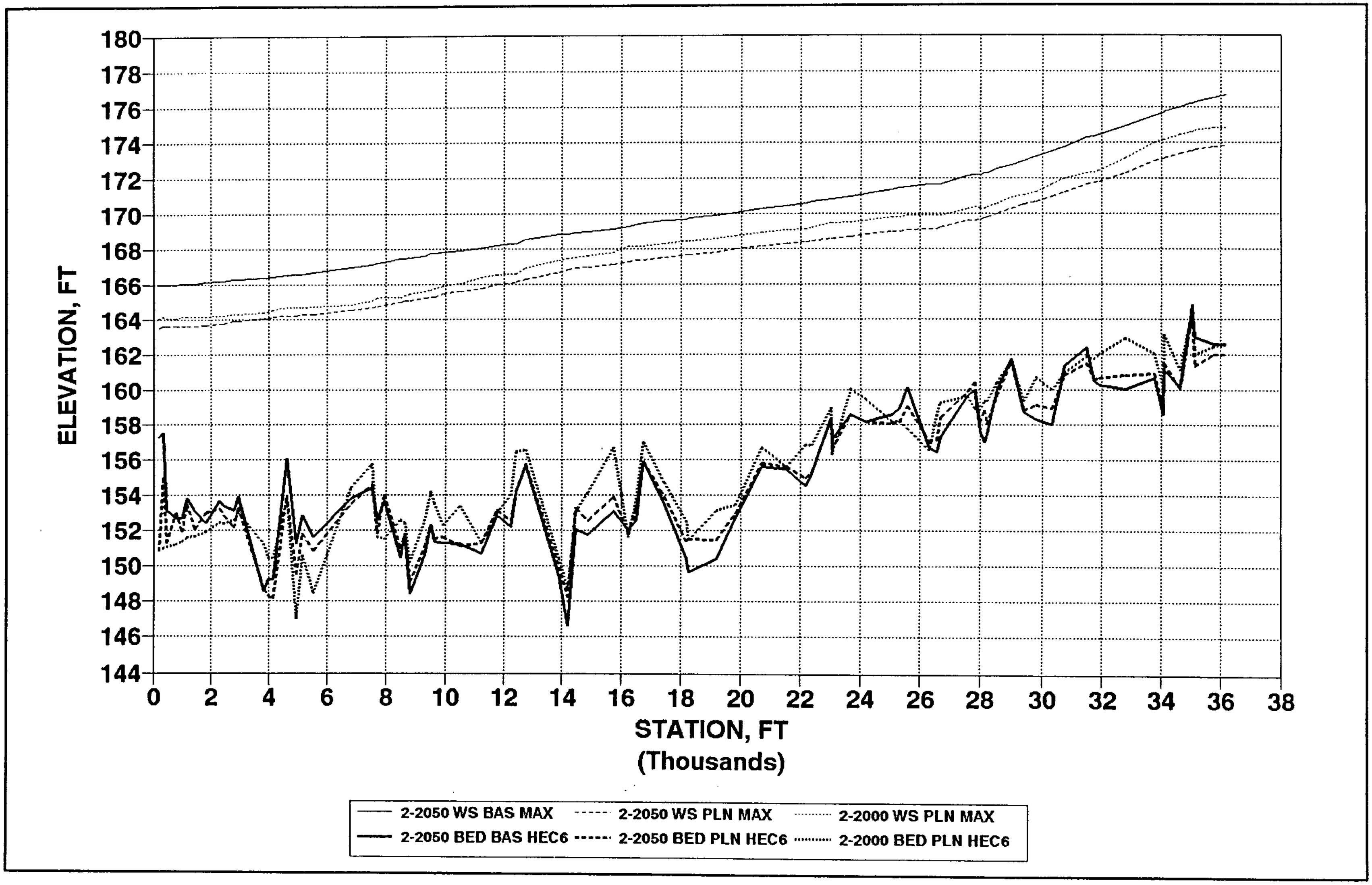

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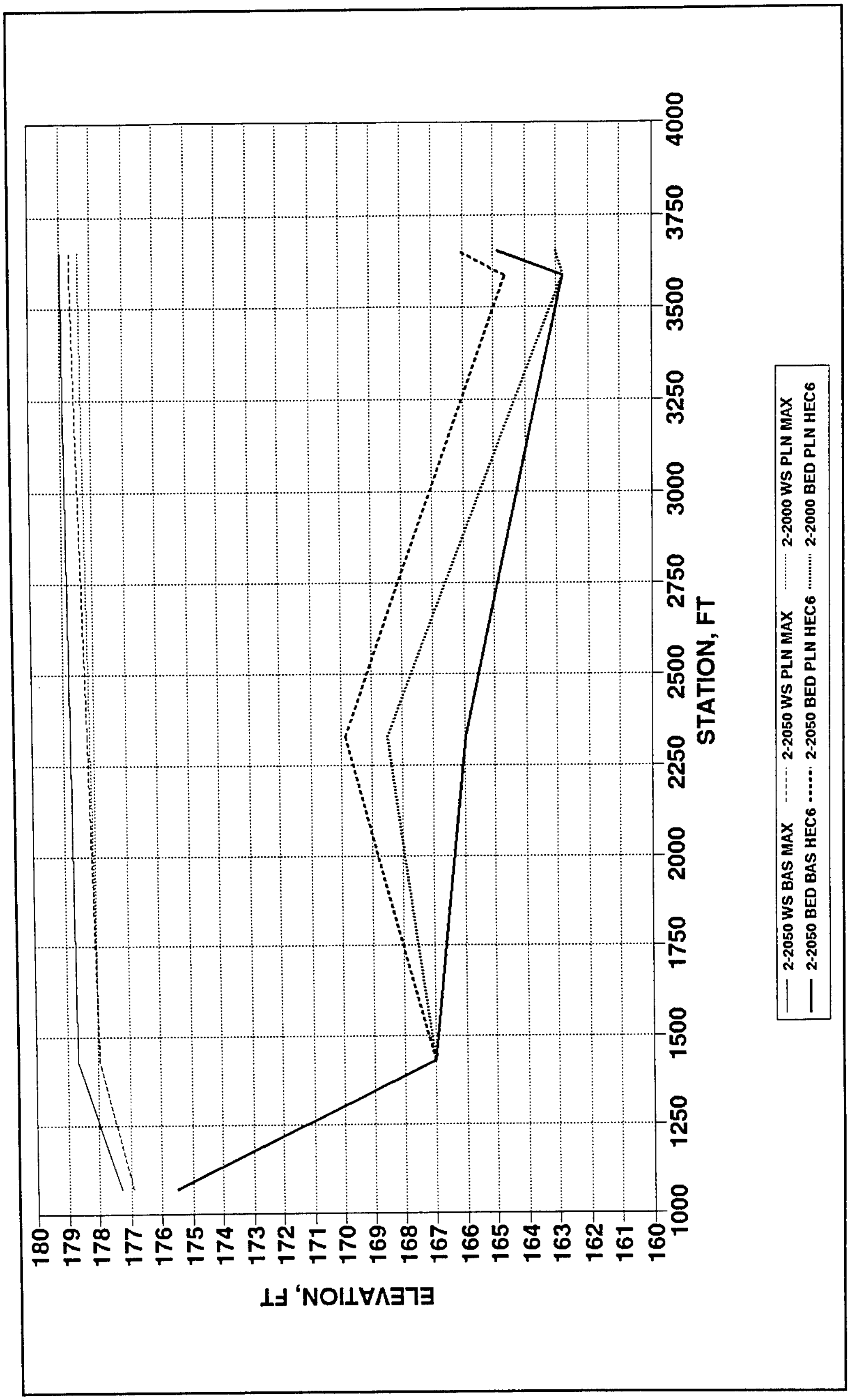

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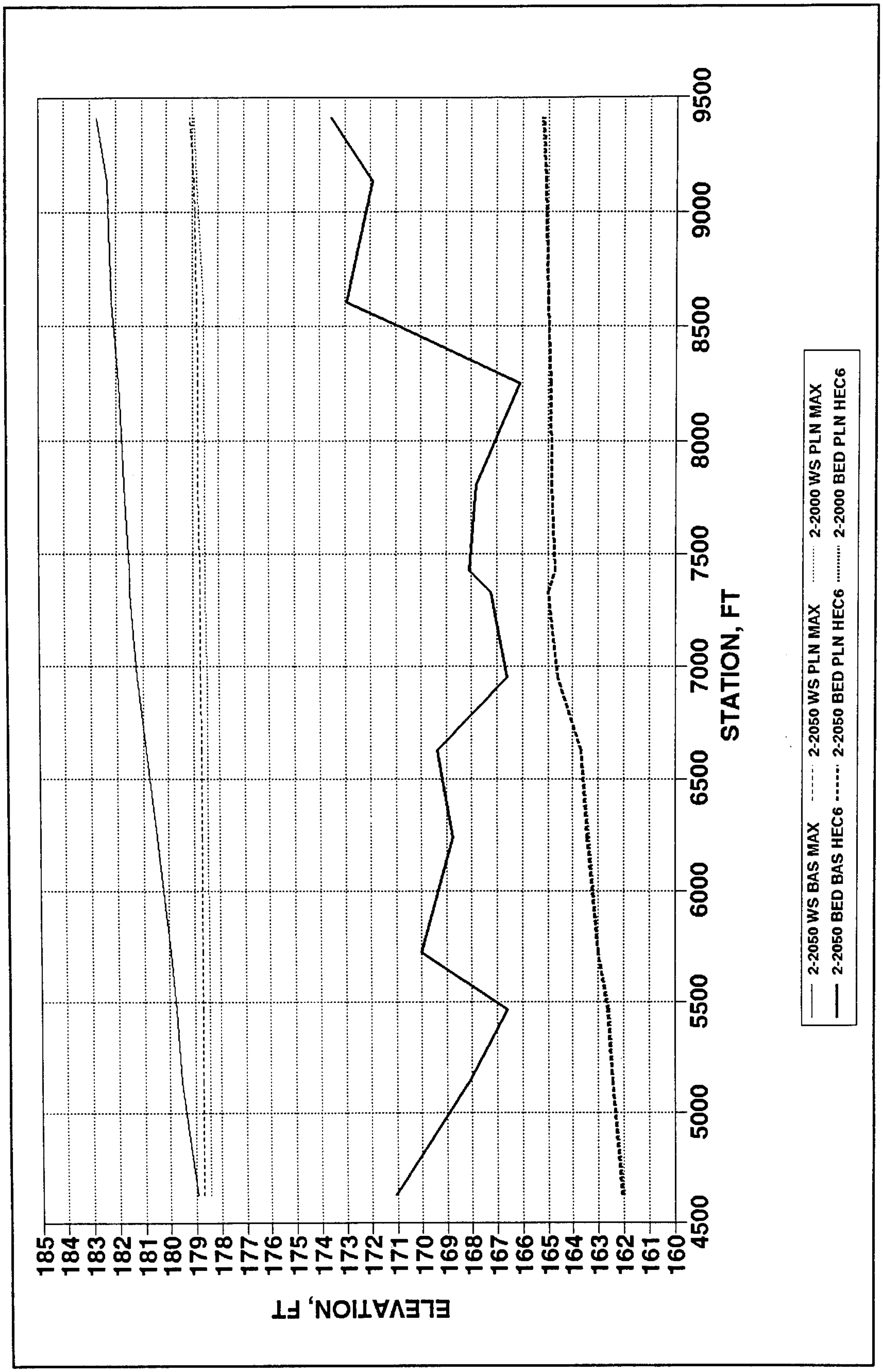

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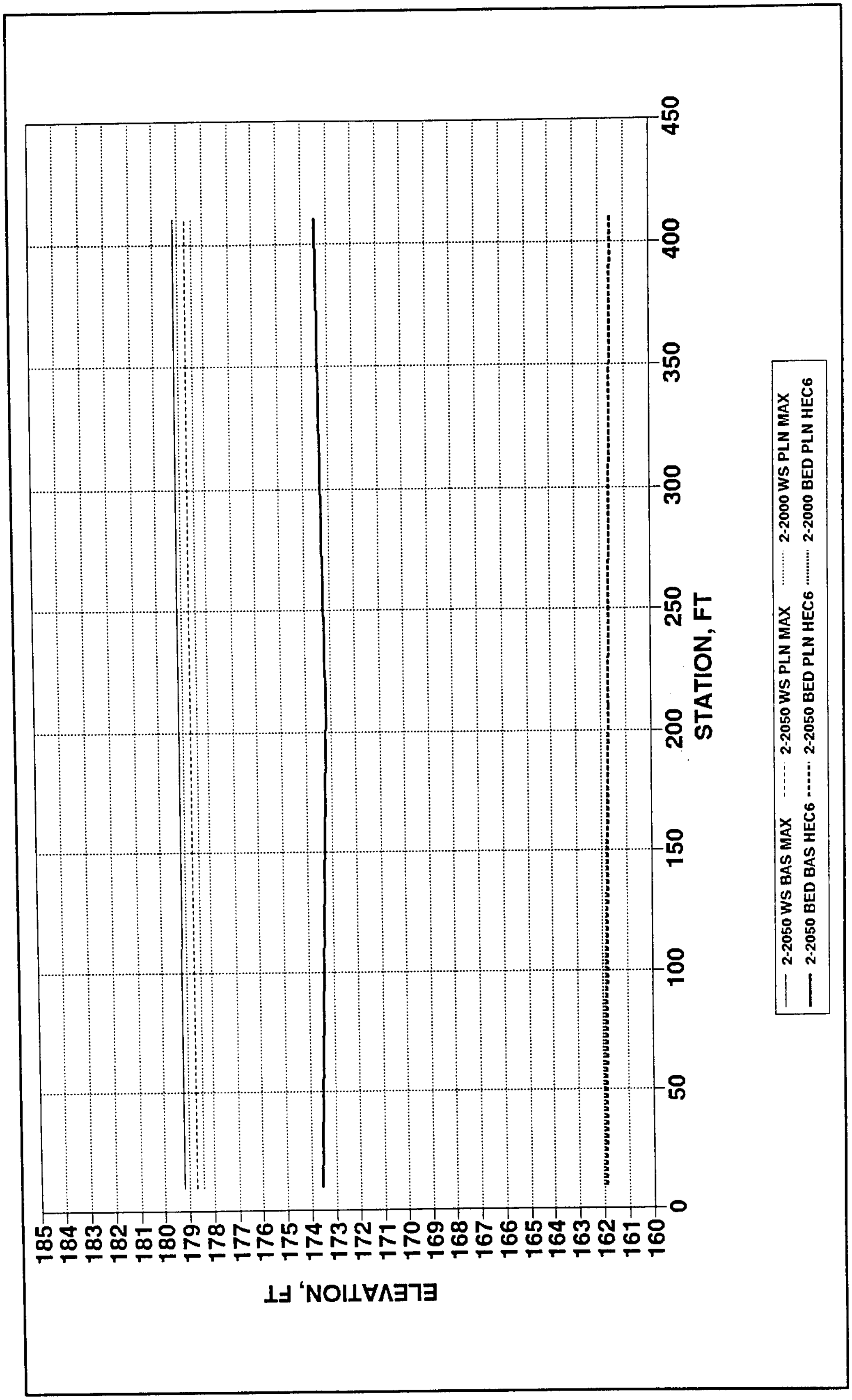
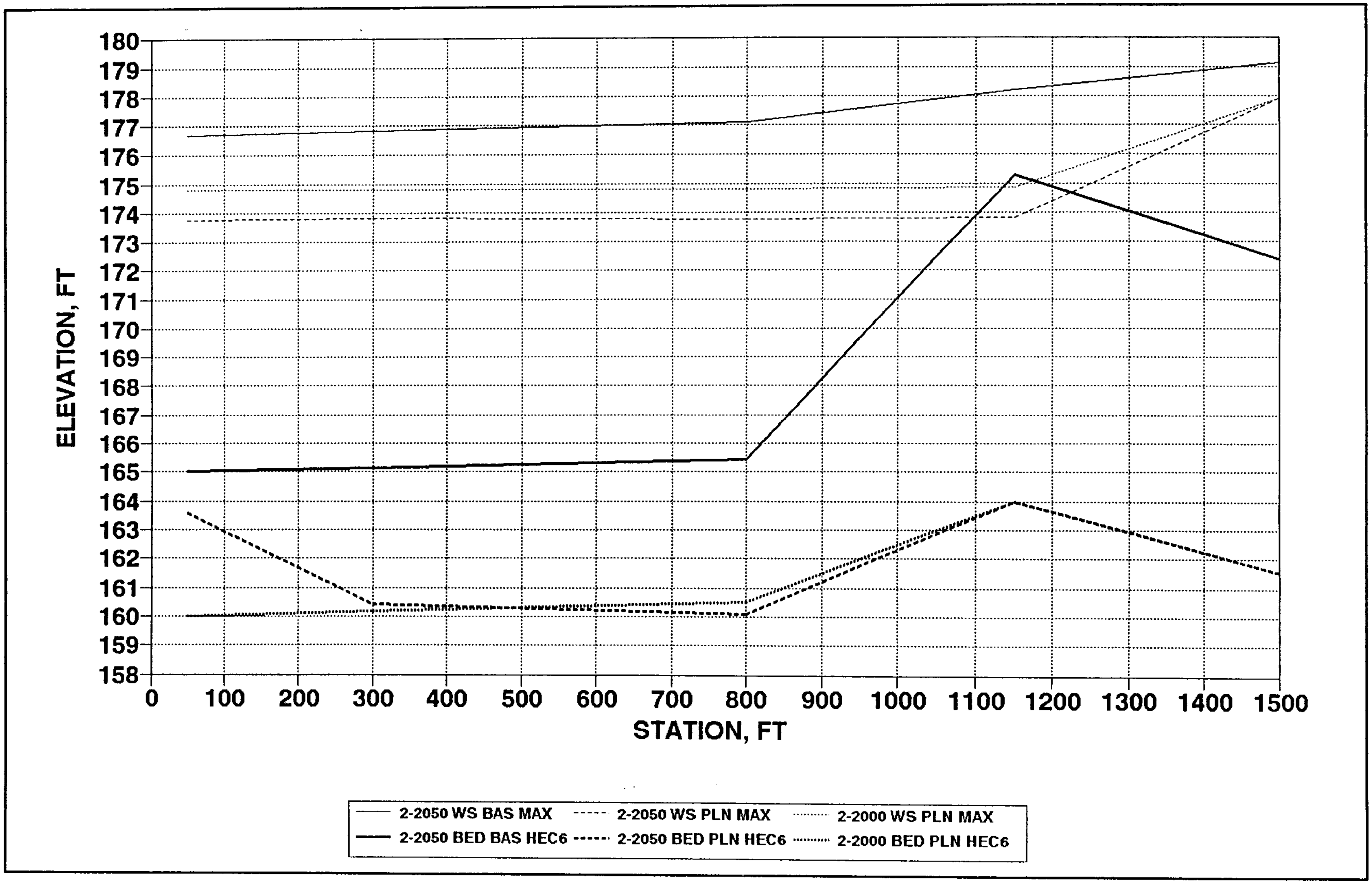


Figure G12. Base and plan long-term simulation with 2-year hydrograph, bed and water-surface profiles, Branch 9



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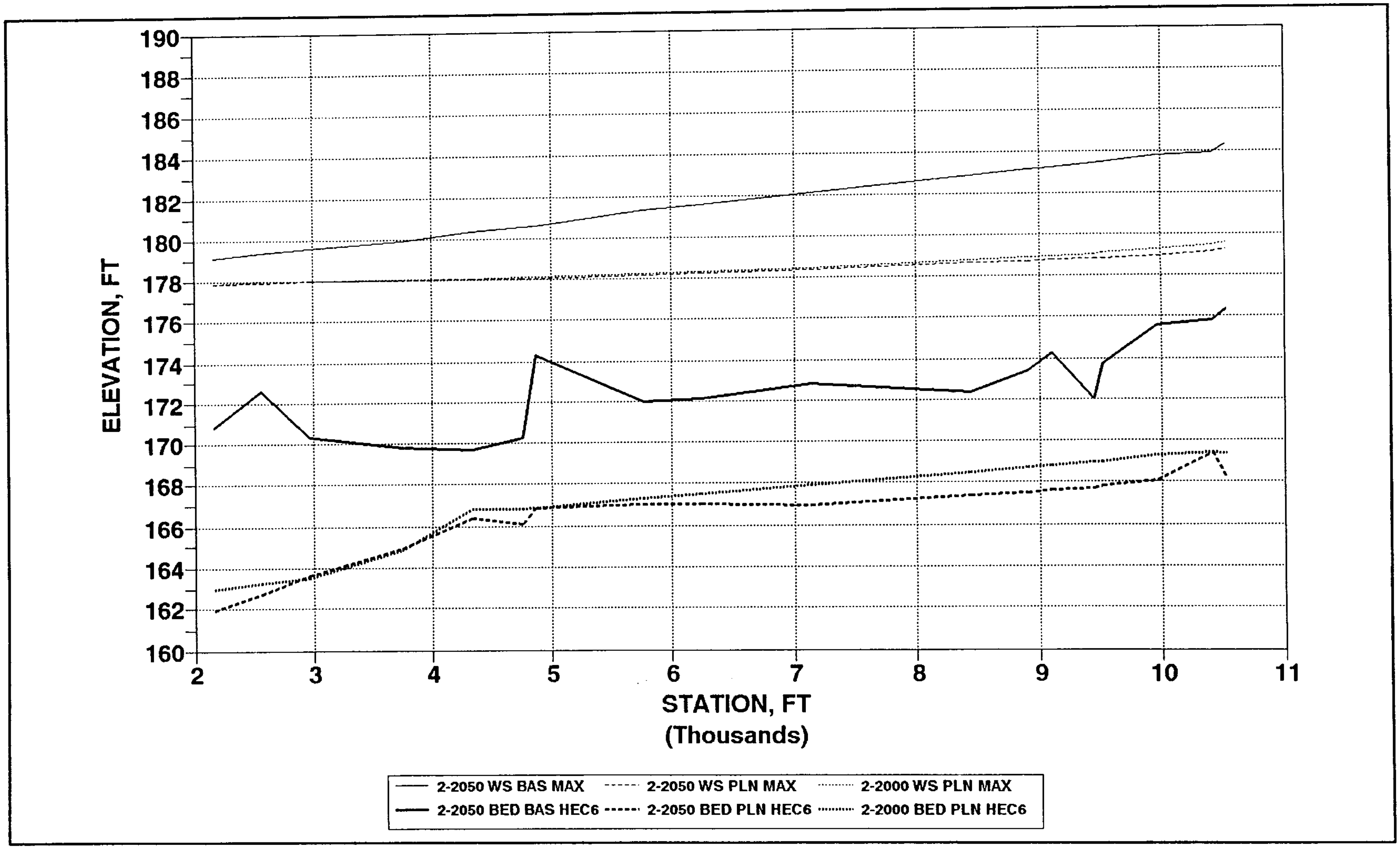
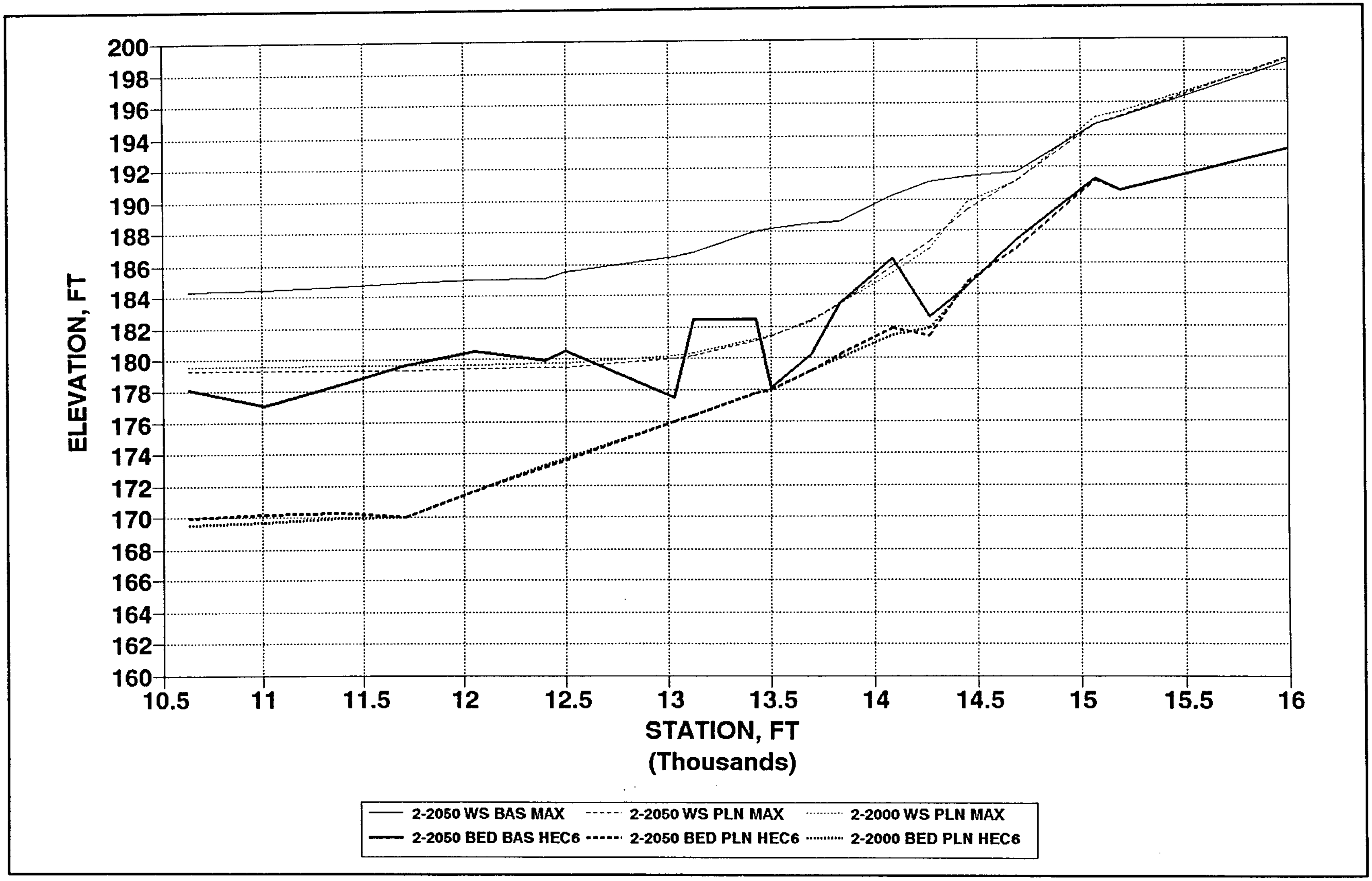


Figure G14. Base and plan long-term simulation with 2-year hydrograph, bed and water-surface profiles, Branch 11, Pequannock River



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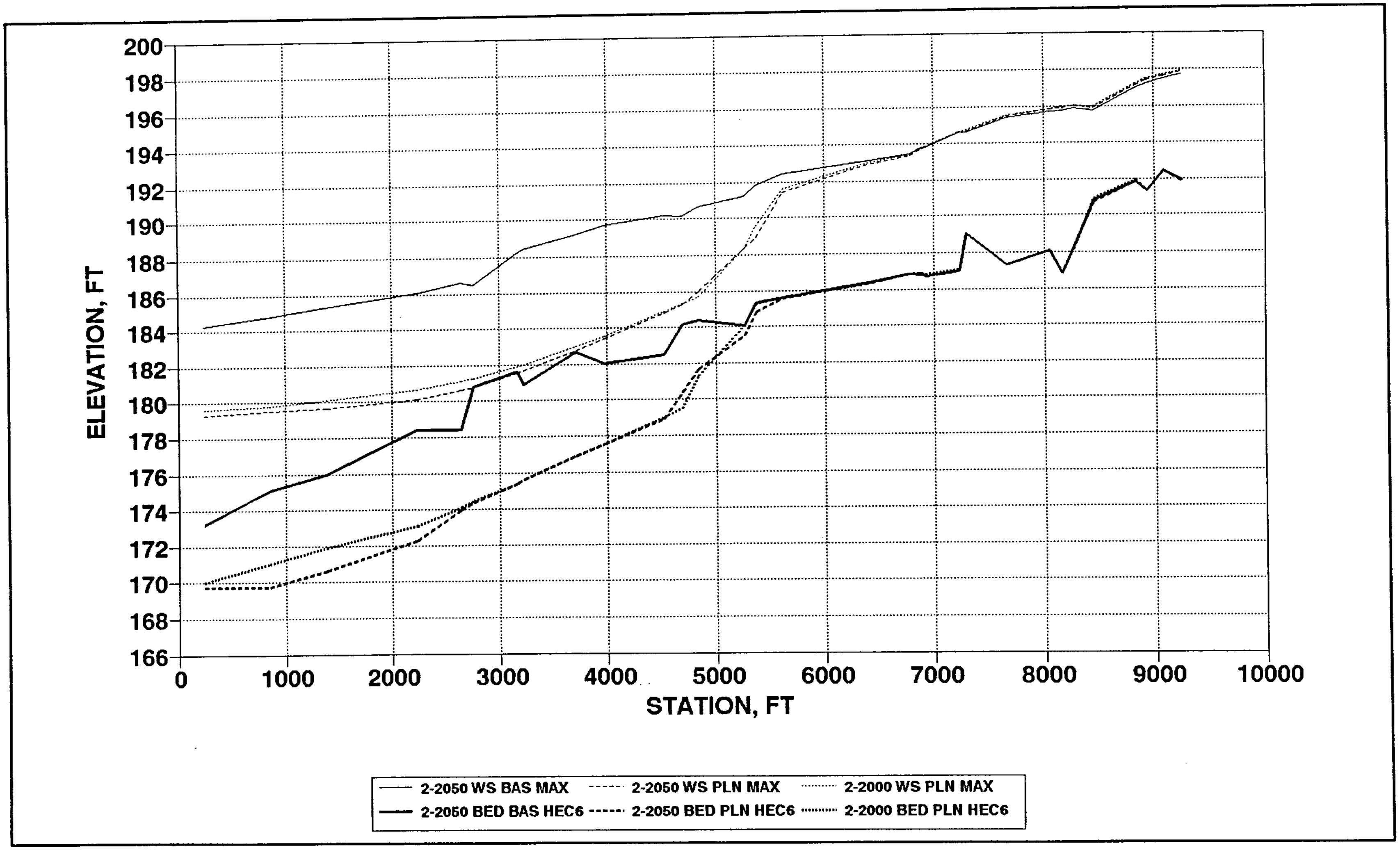

Figure G16. Base and plan long-term simulation with 2-year hydrograph, bed and water-surface profiles, Branch 13, Wanaque River

Appendix H Base and Plan Long-Term Simulation with 25-year Hydrograph

This appendix contains graphs for river branches 1 to 13 showing the differences between base and plan beds and base and plan maximum water surfaces with a 14-day-duration 25-year hydrograph superimposed on the 50-year simulation (year 2050).

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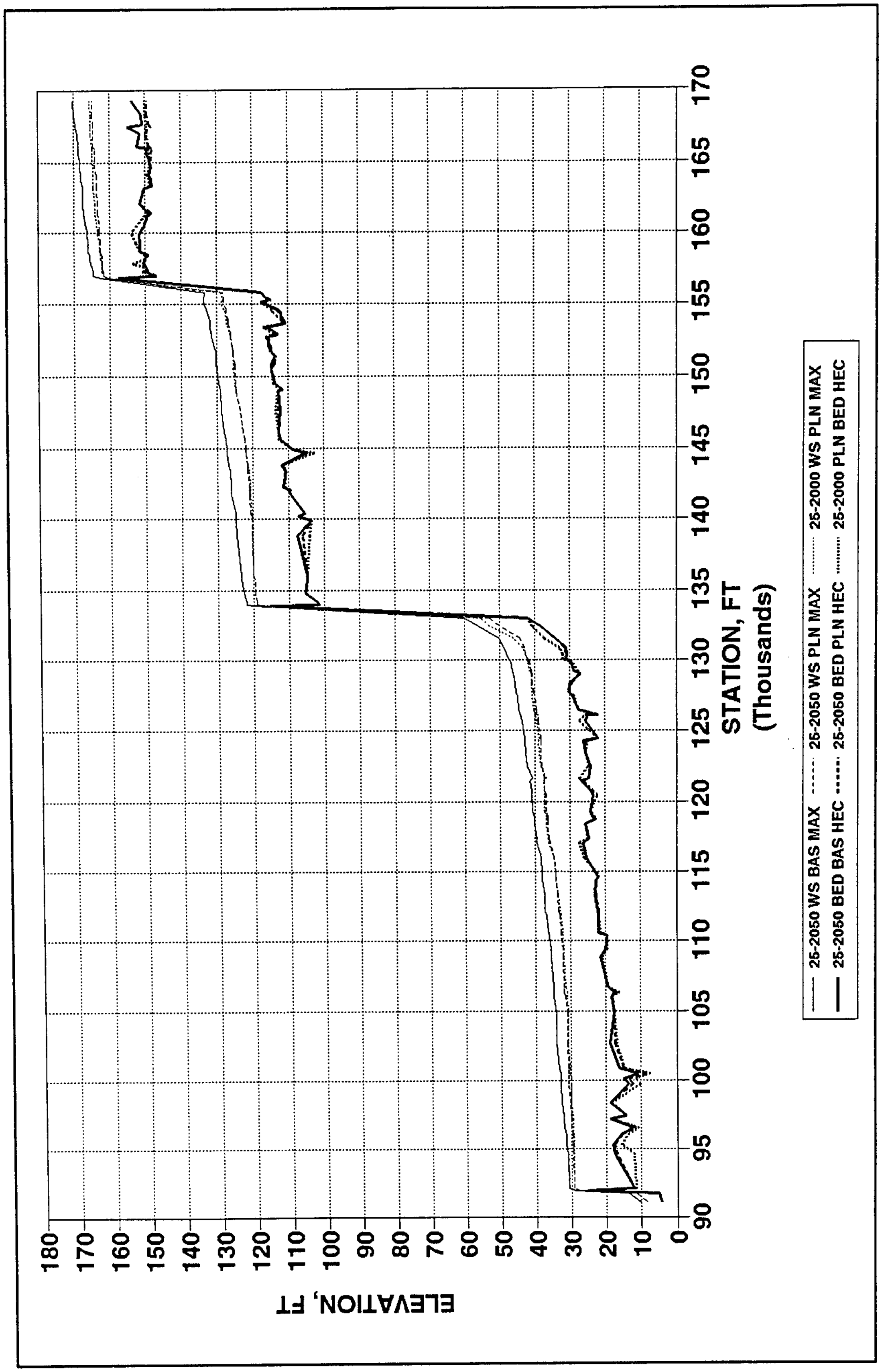
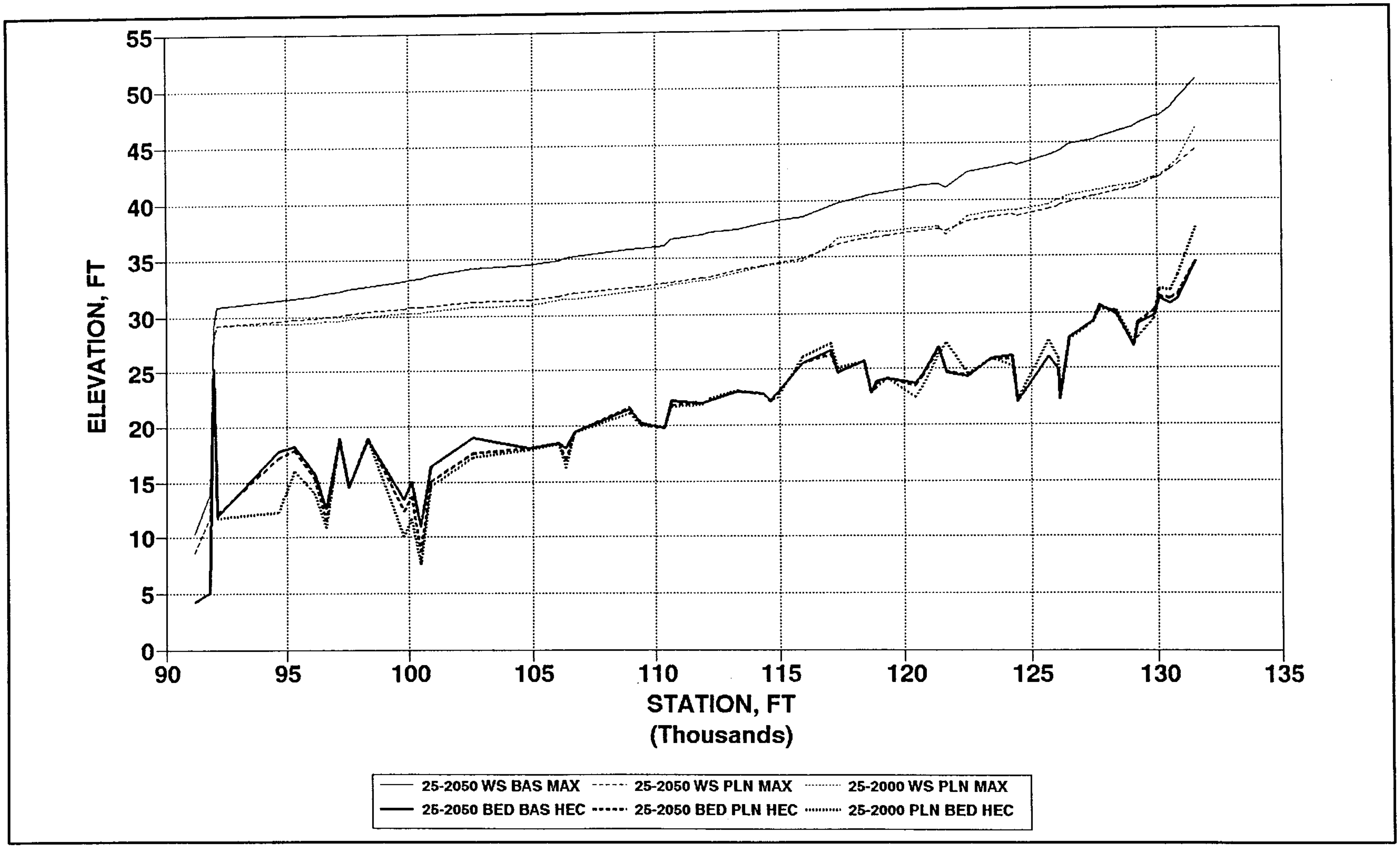


Figure H1. Base and plan long-term simulation with 25-year hydrograph, bed and water-surface profiles, Branch 1, Passaic River



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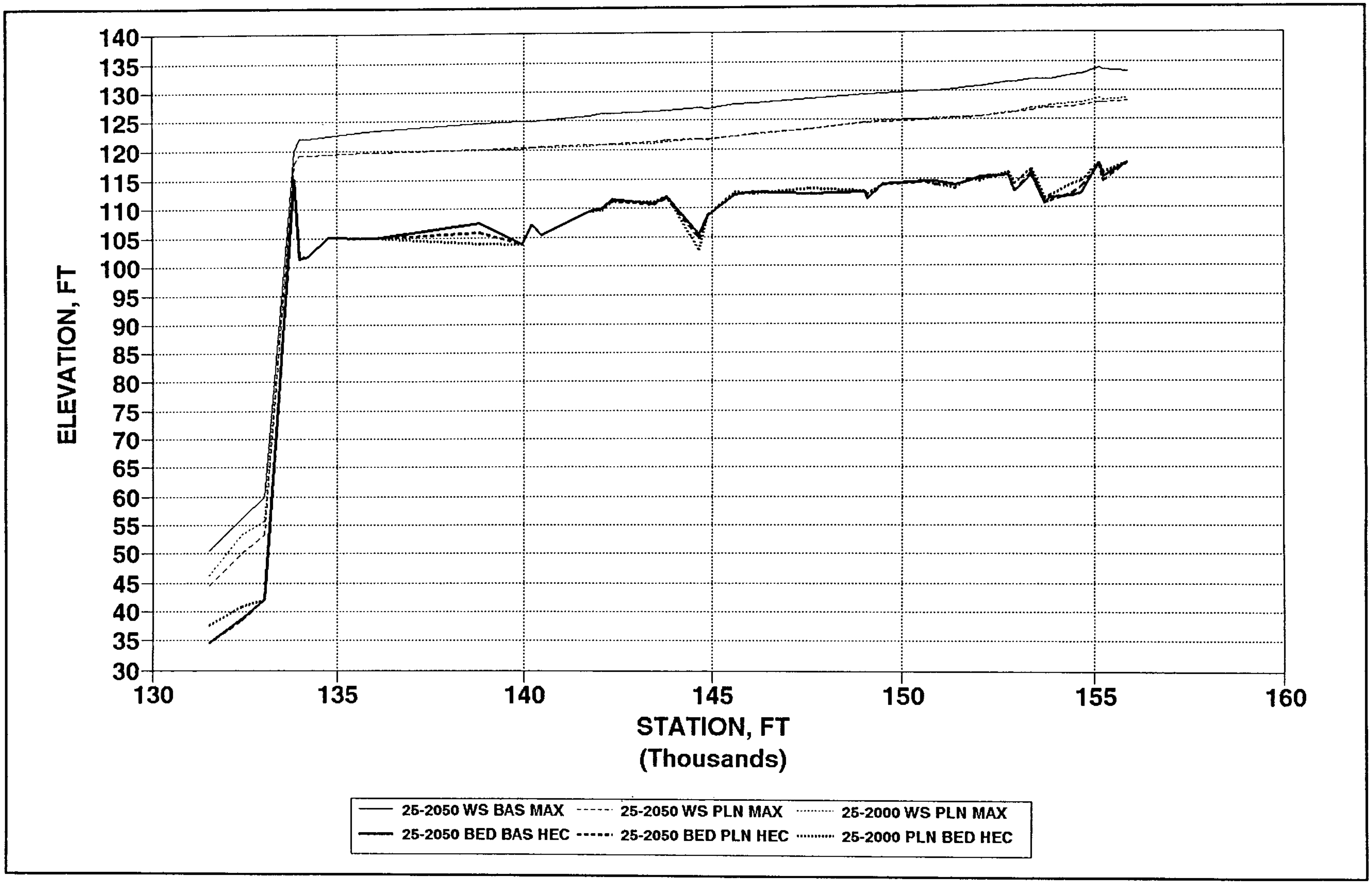
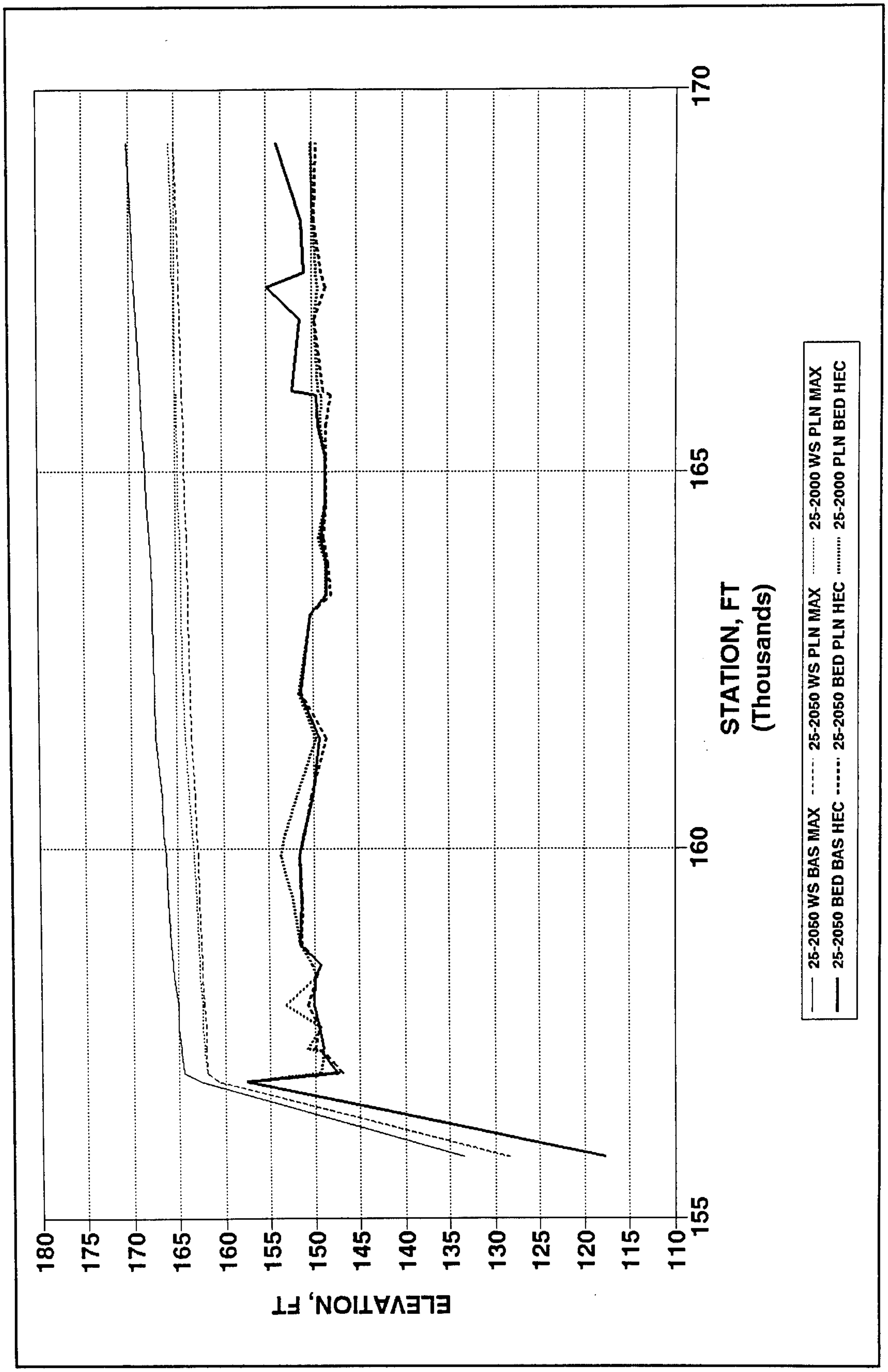


Figure H3. Base and plan long-term simulation with 25-year hydrograph, bed and water-surface profiles, Branch 1, Passaic River, S.U.M. Dam



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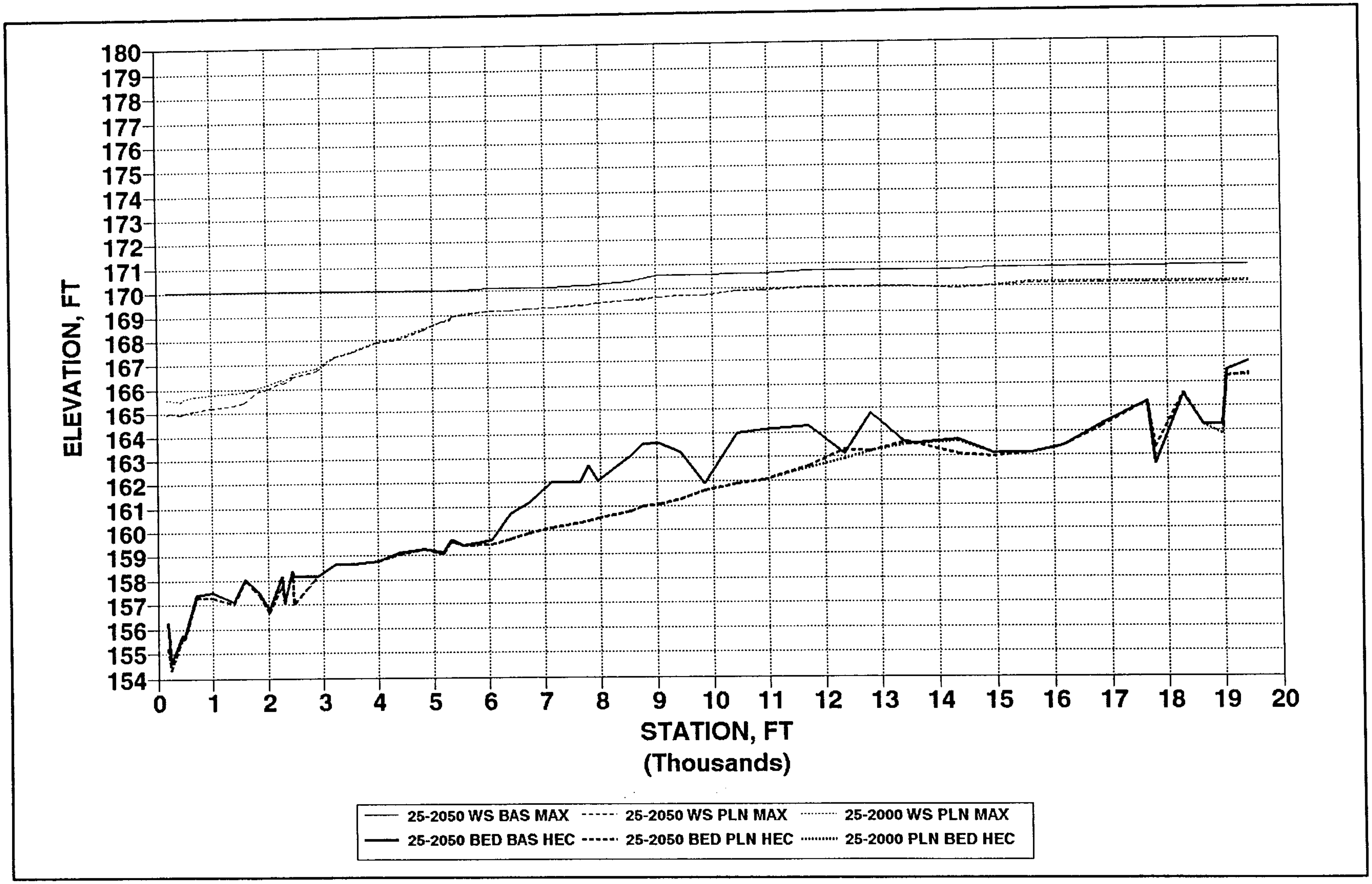

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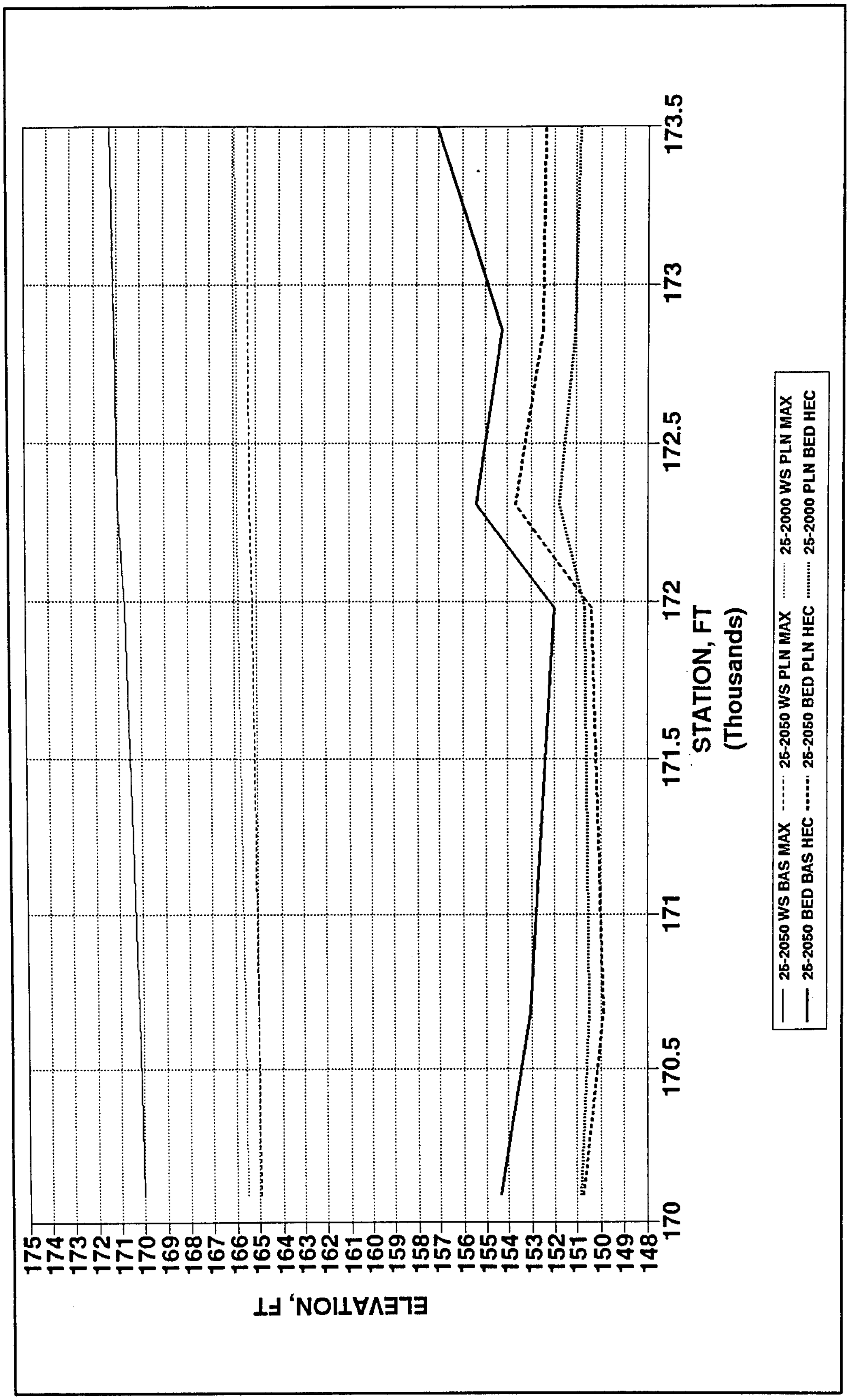

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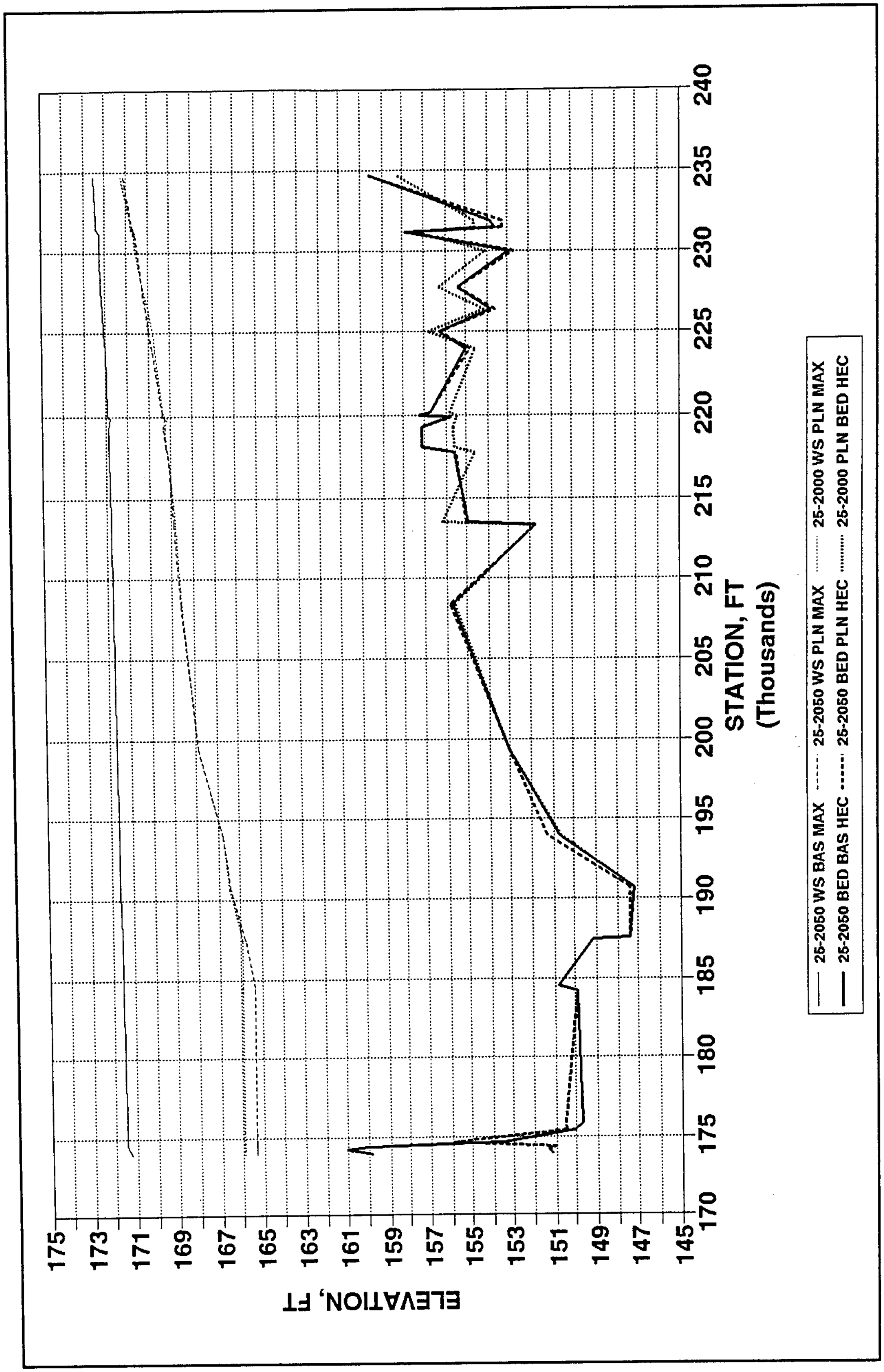

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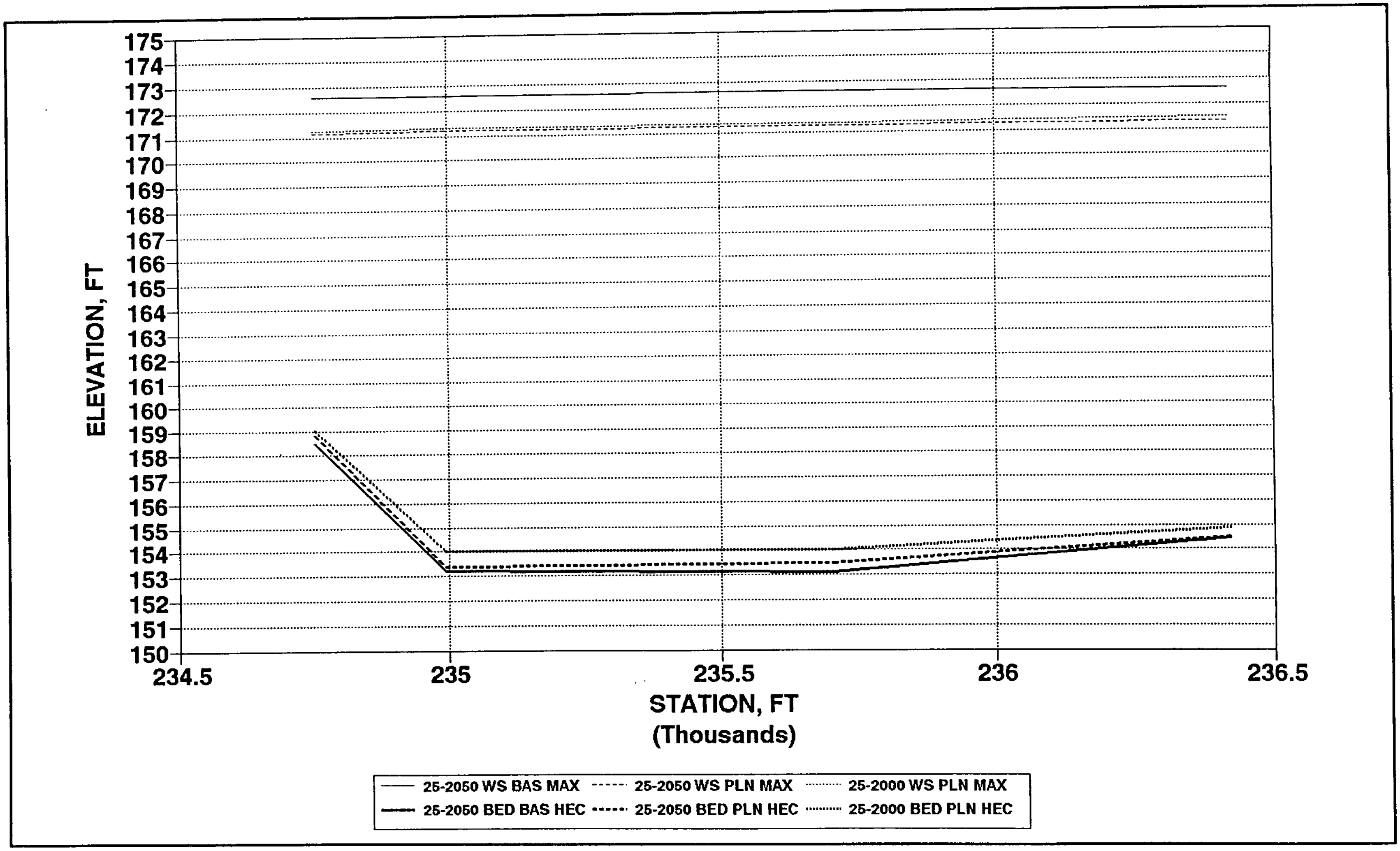

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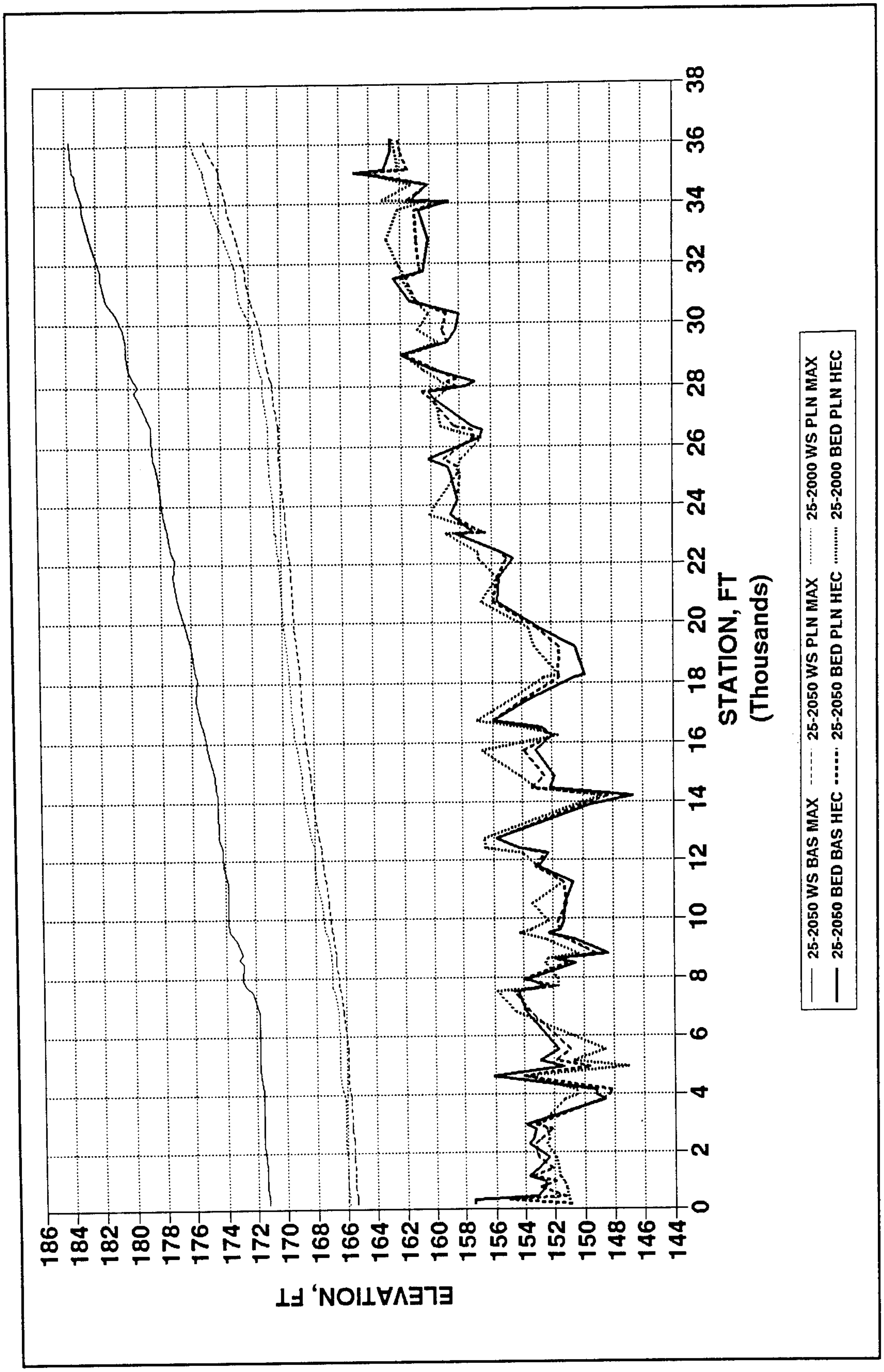

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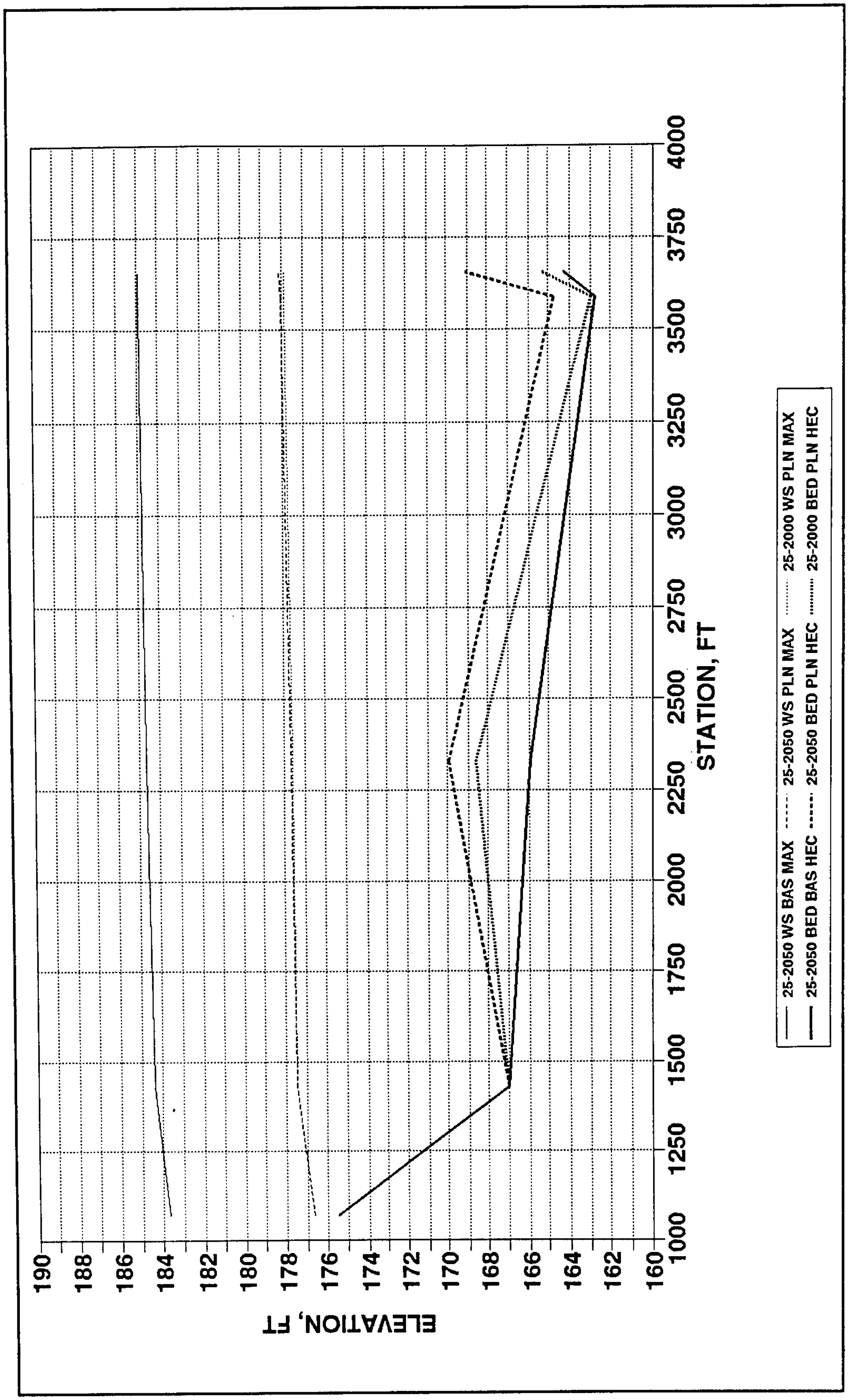

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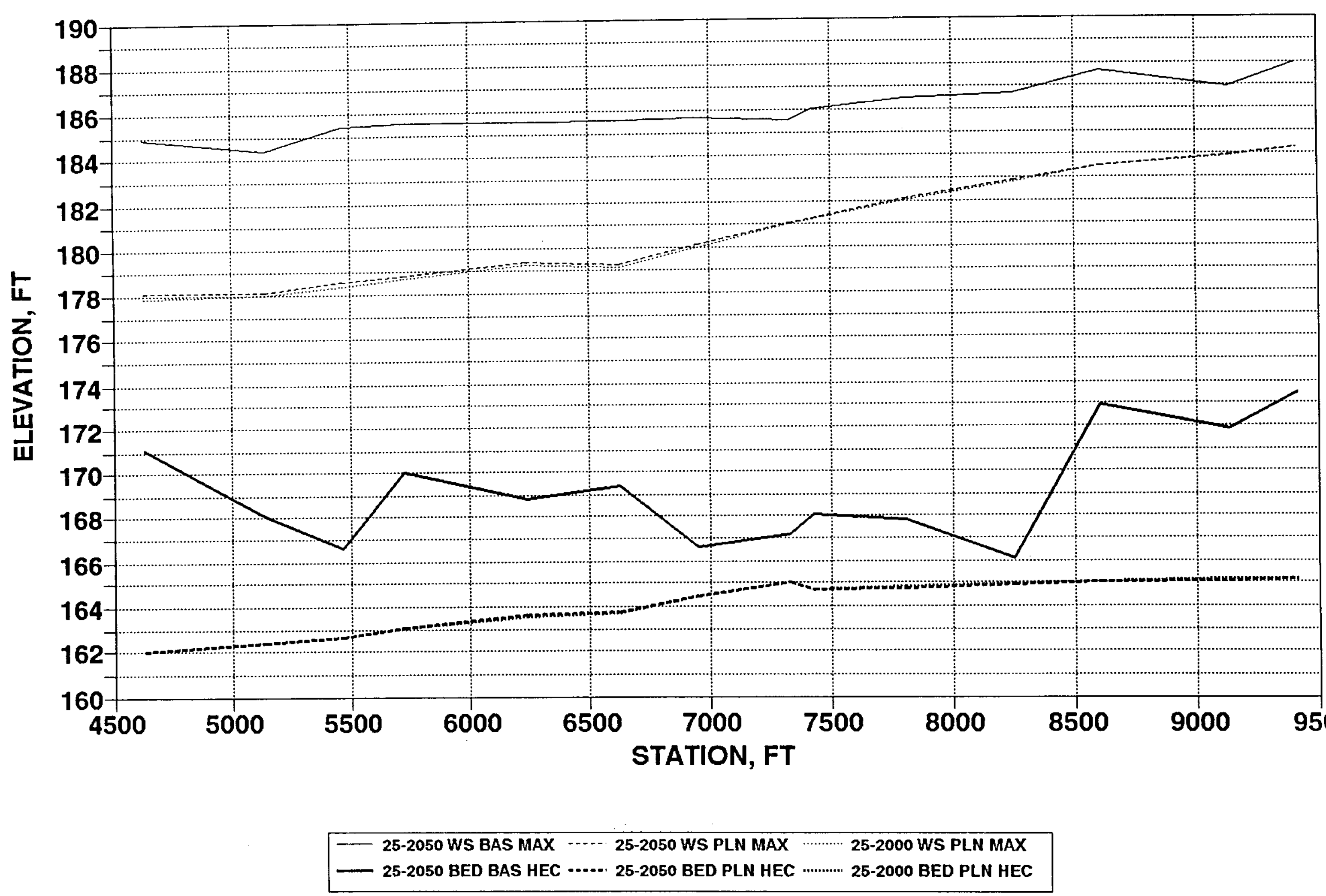

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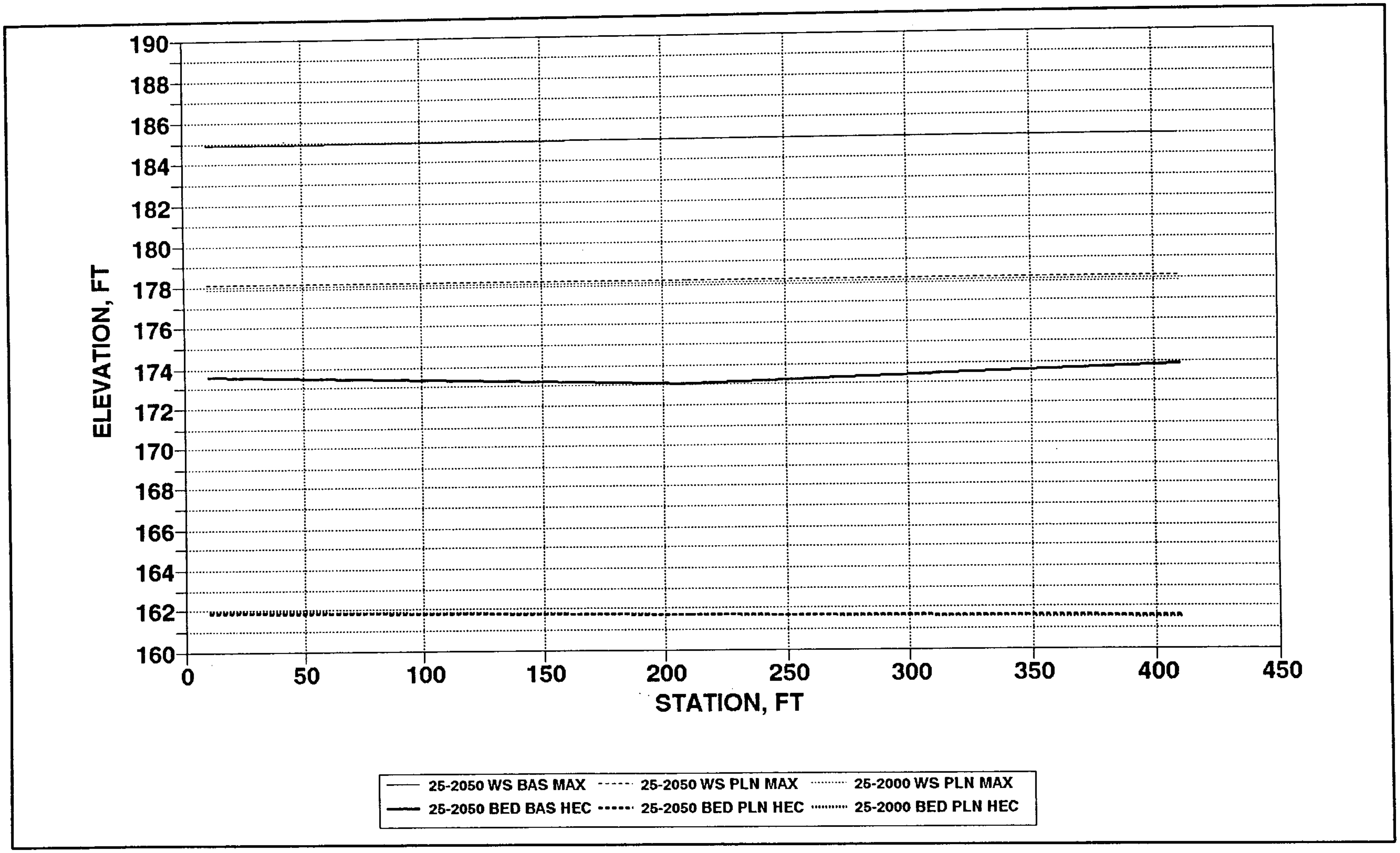

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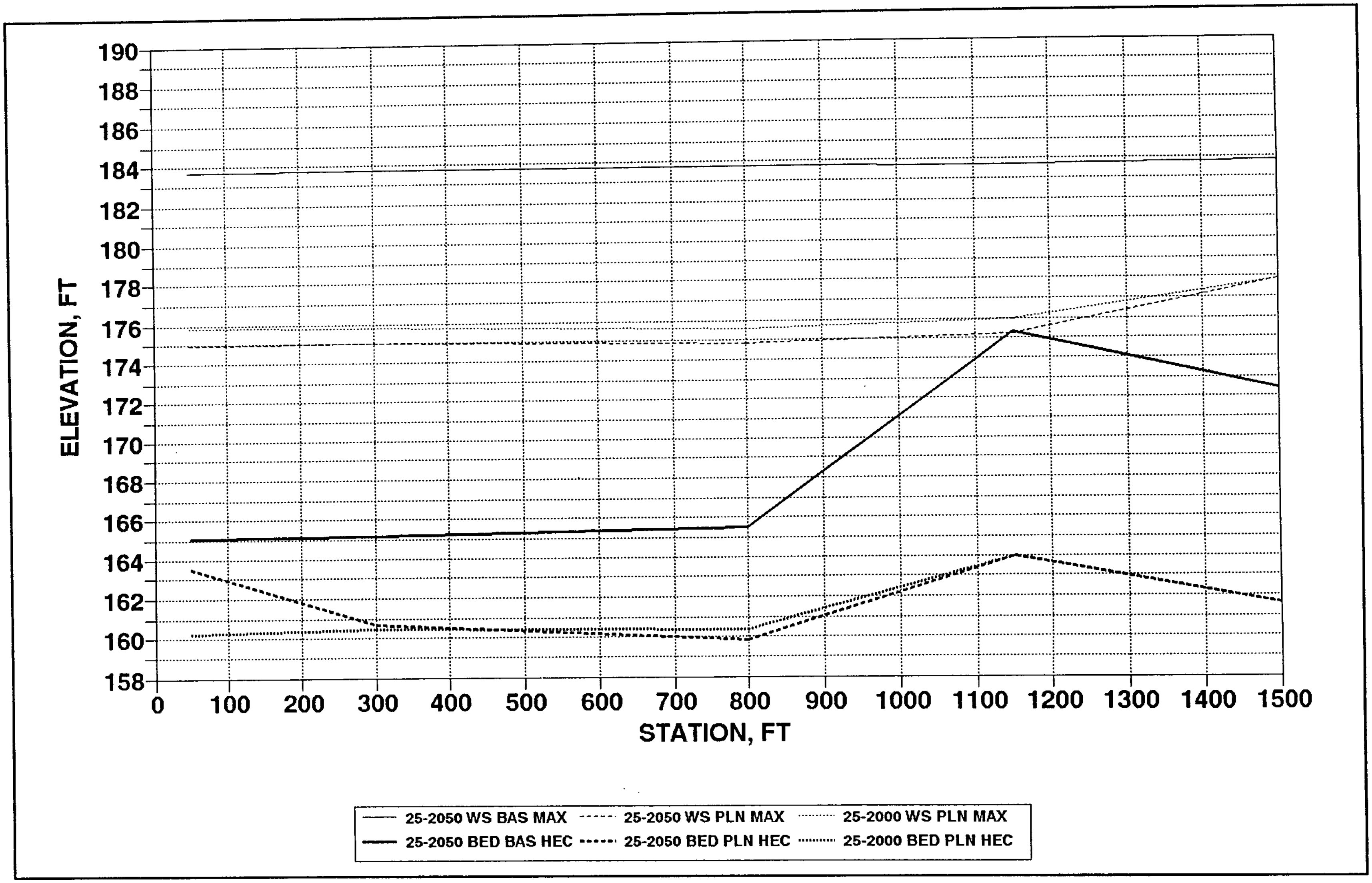

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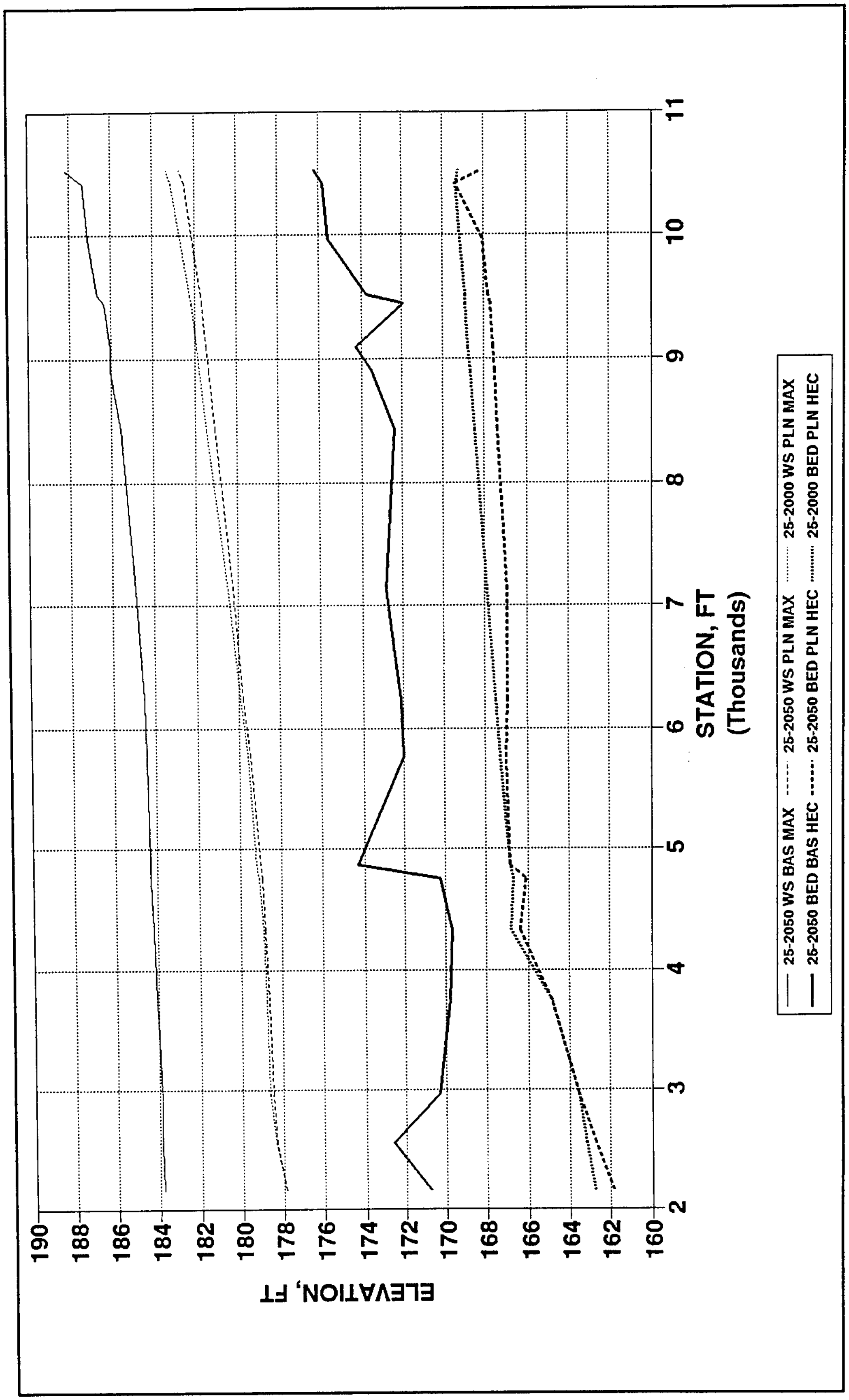

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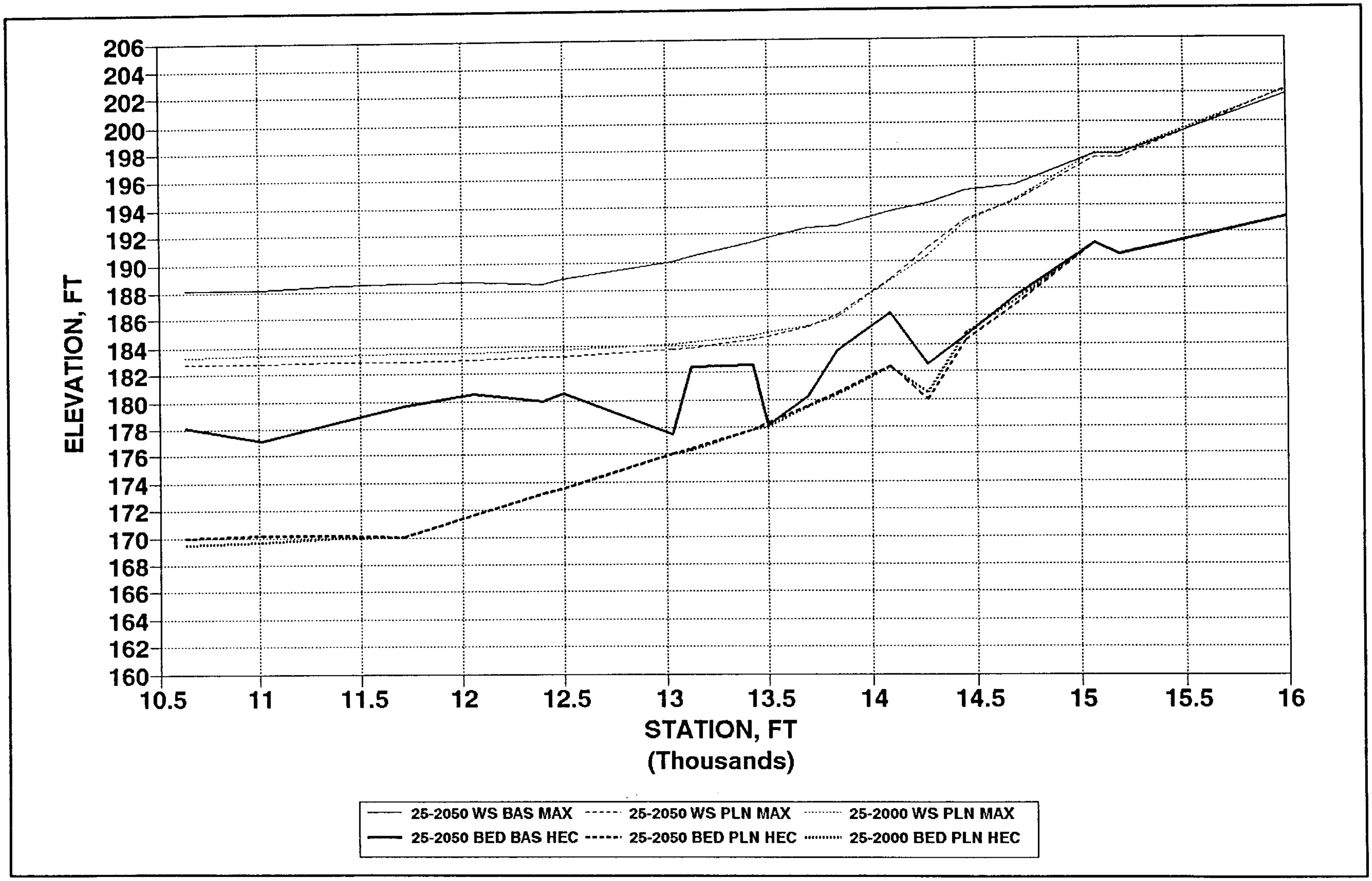

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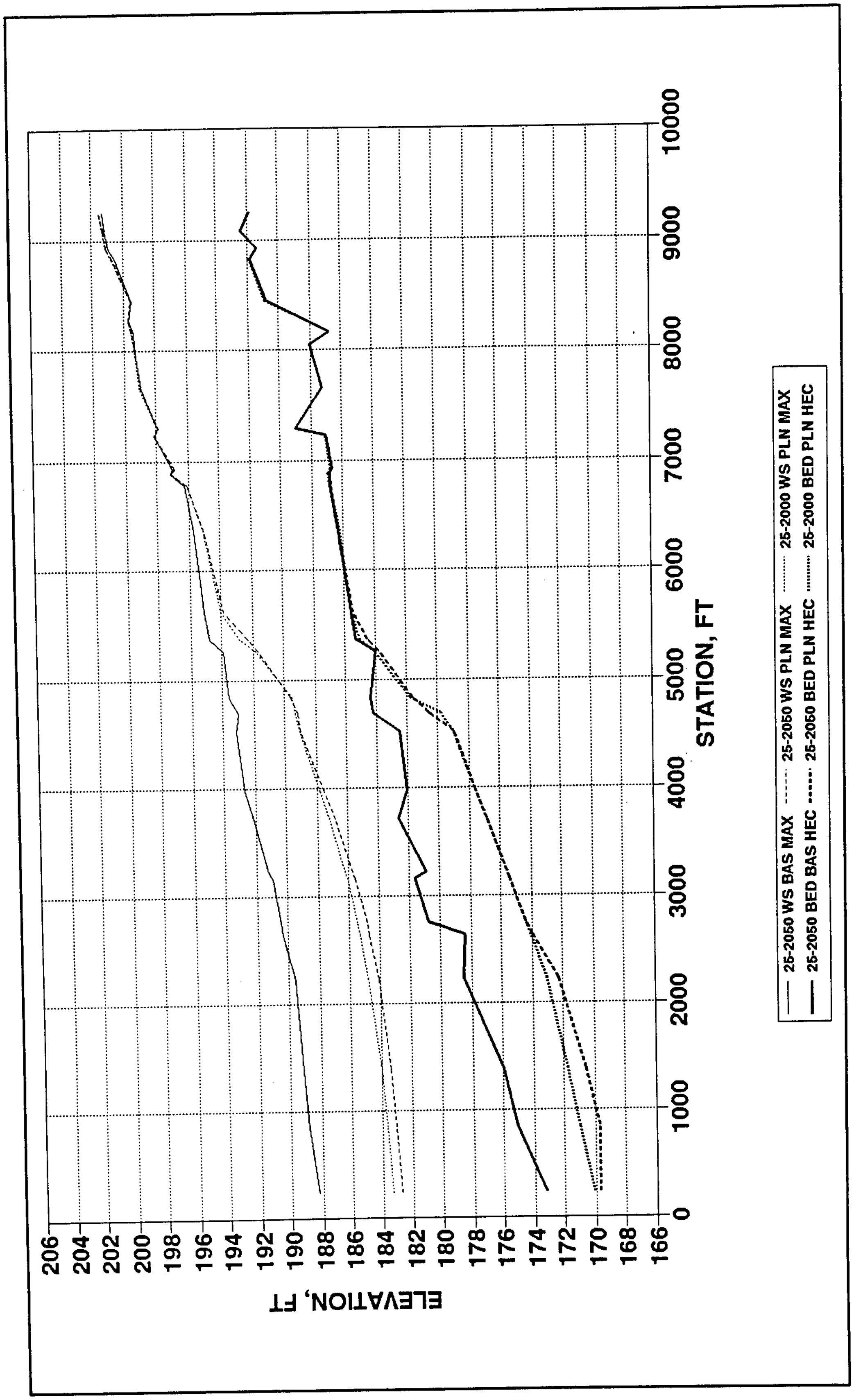

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Appendix I
Base and Plan Long-Term Simulation with 100-year Hydrograph

This appendix contains graphs for river branches 1 to 13 showing the differences between base and plan beds and base and plan maximum water surfaces with a 14-day-duration 100-year hydrograph superimposed on the 50-year simulation (year 2050).

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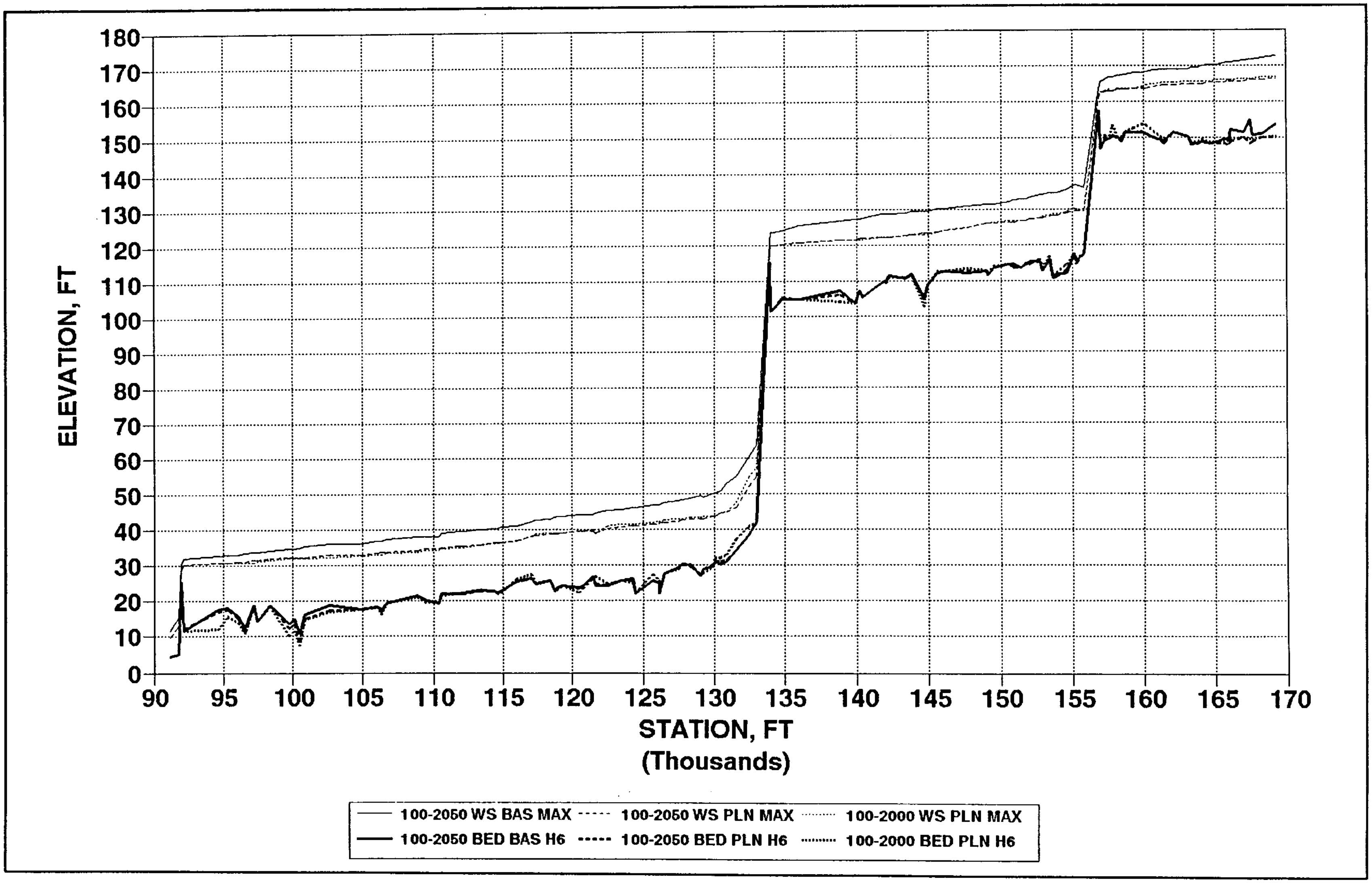

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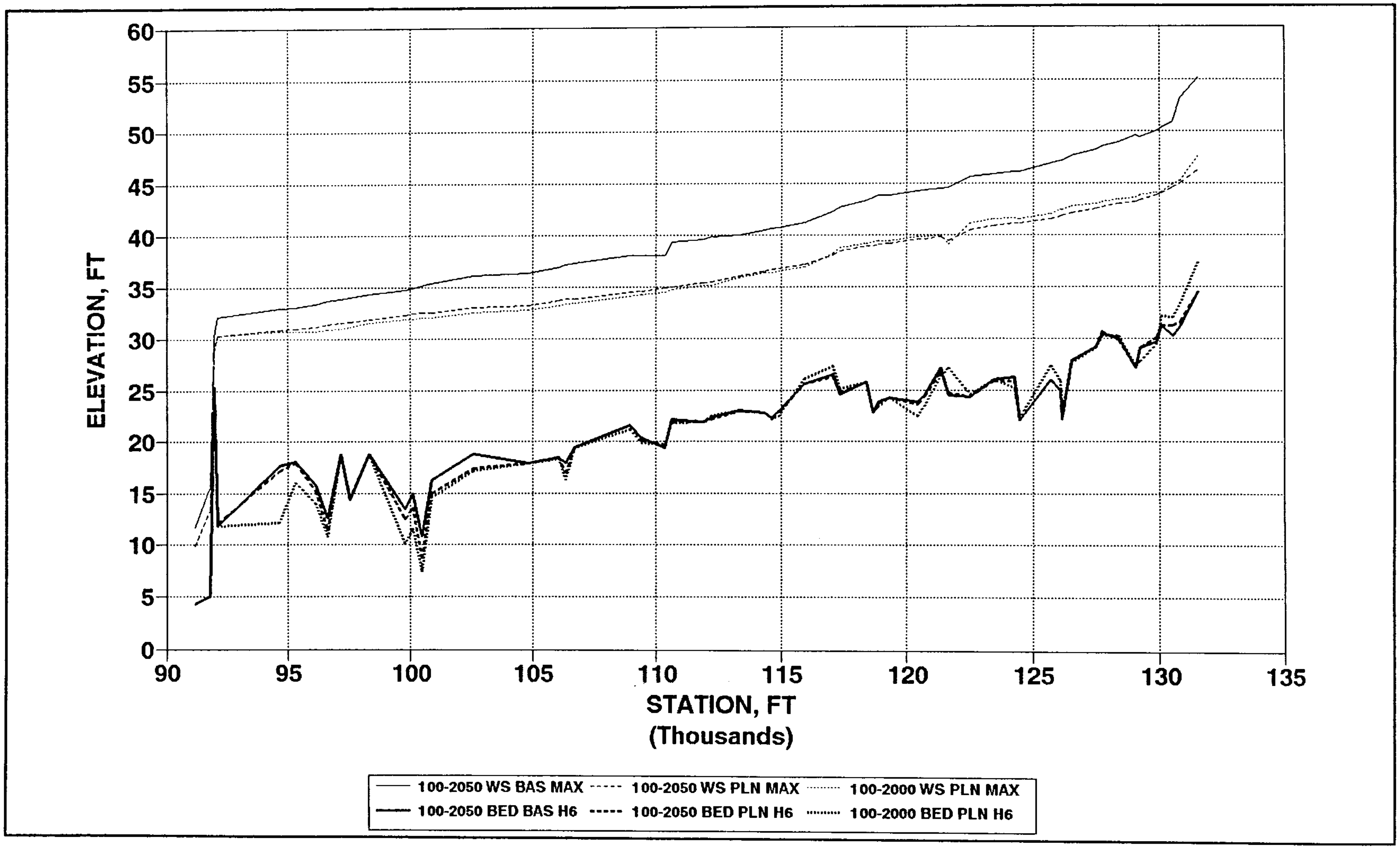

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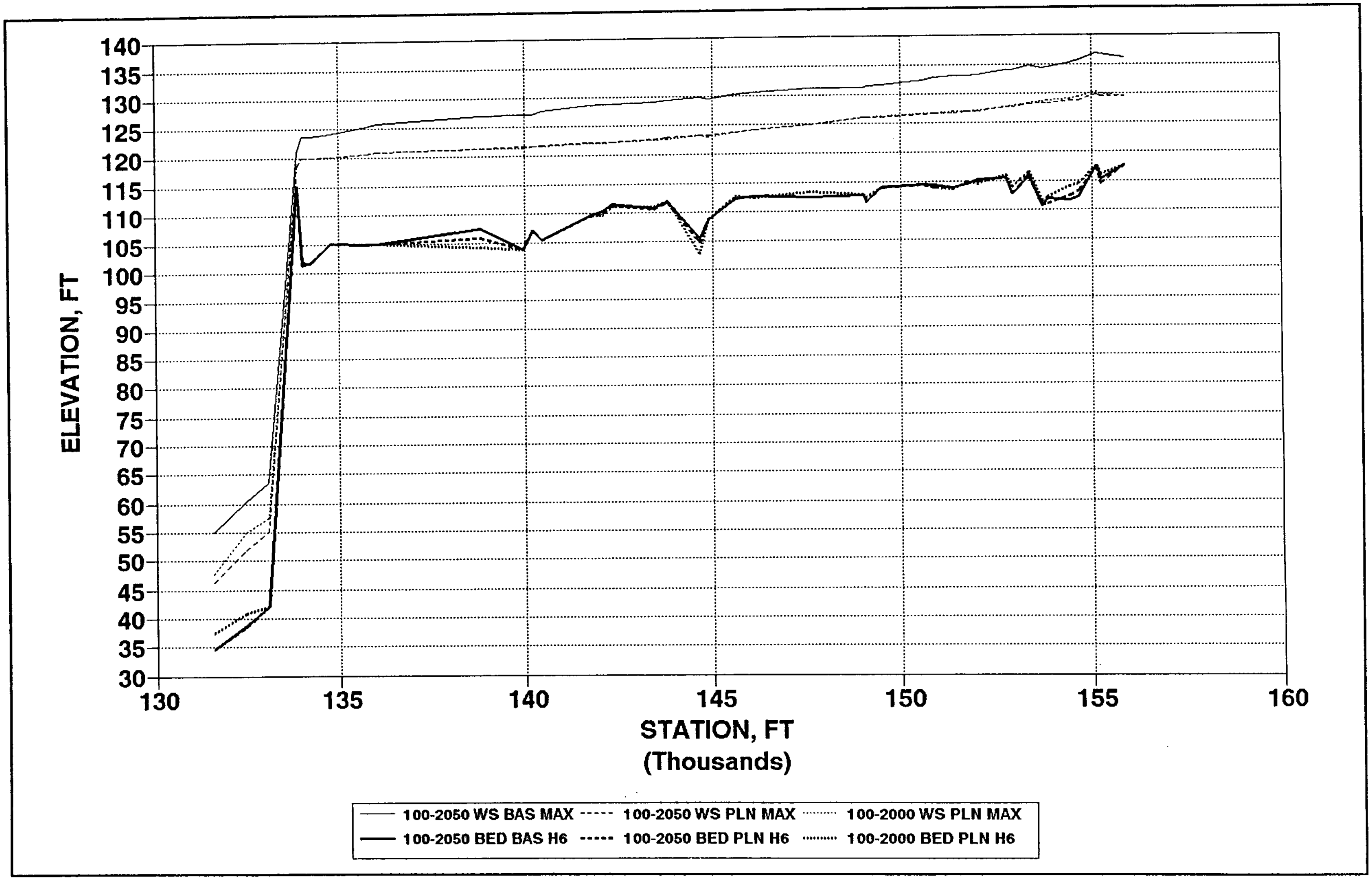

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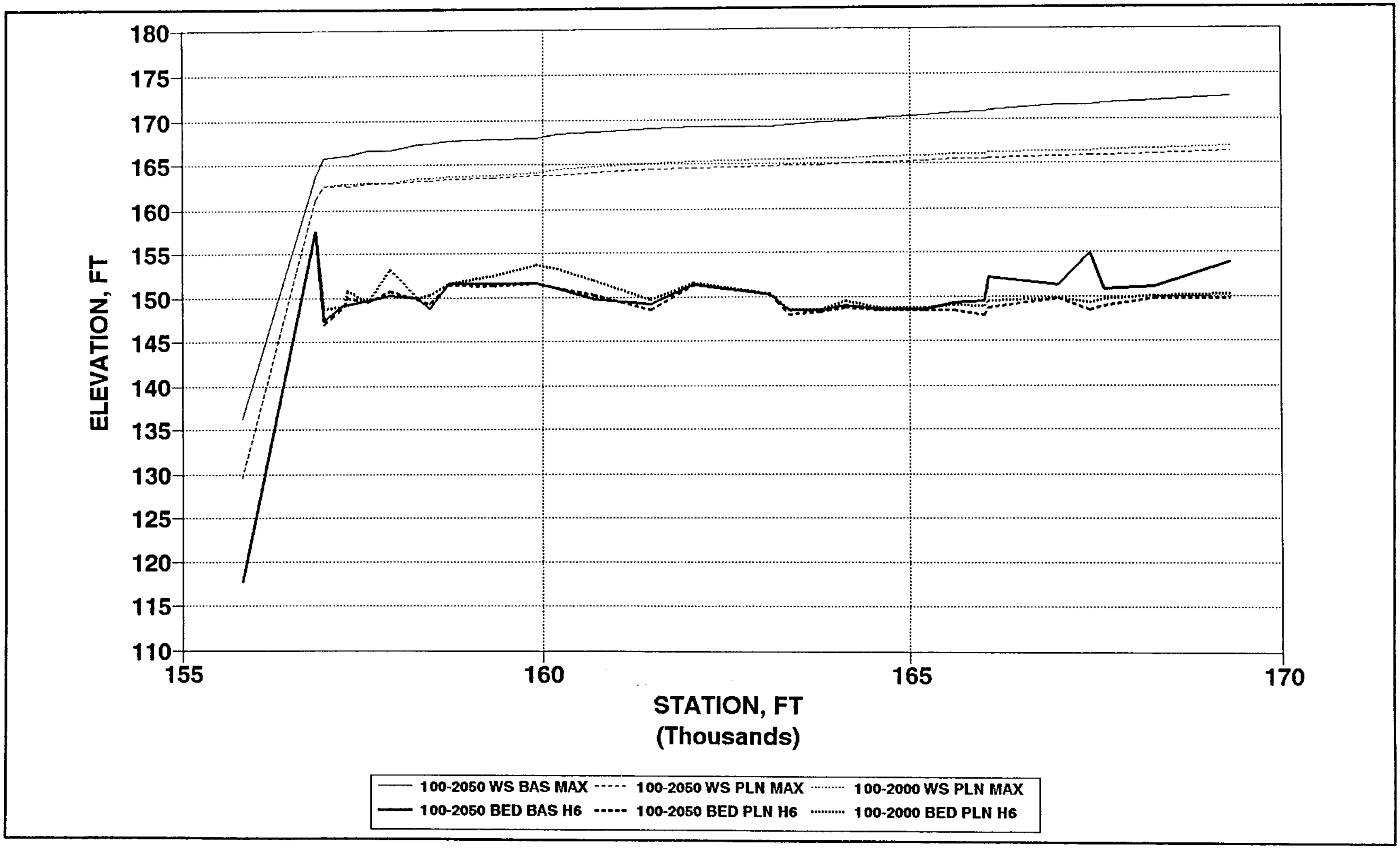

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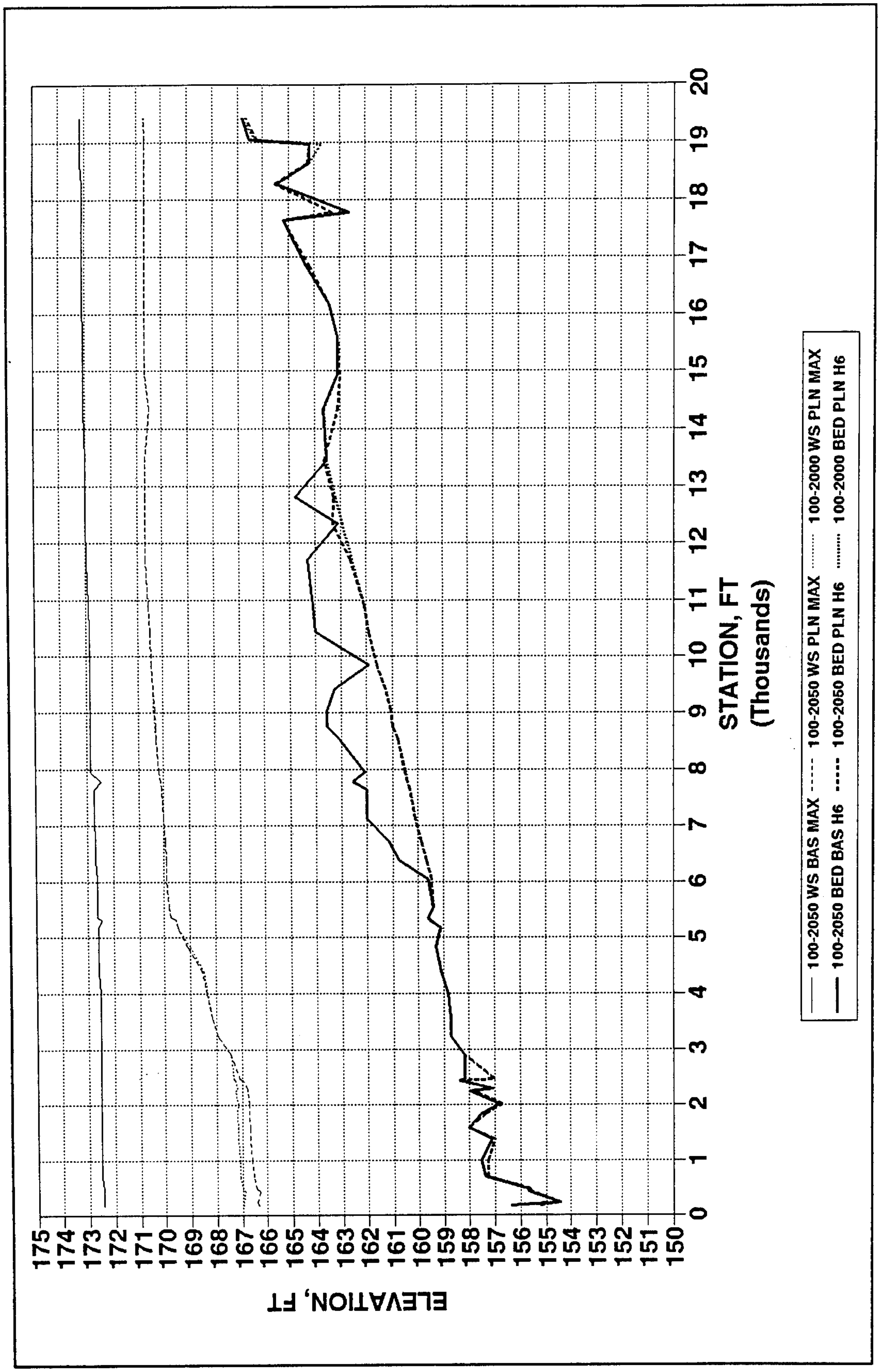

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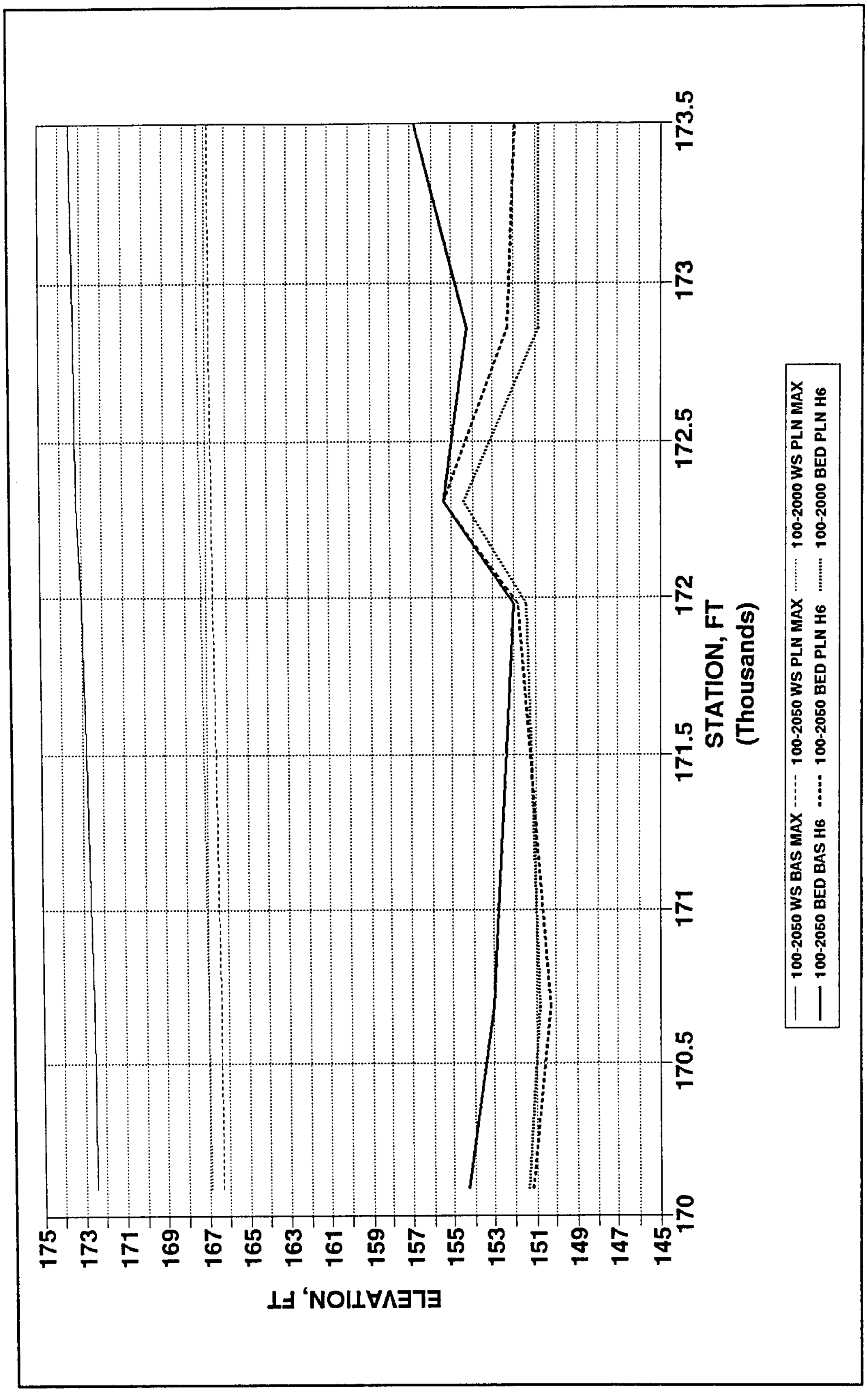

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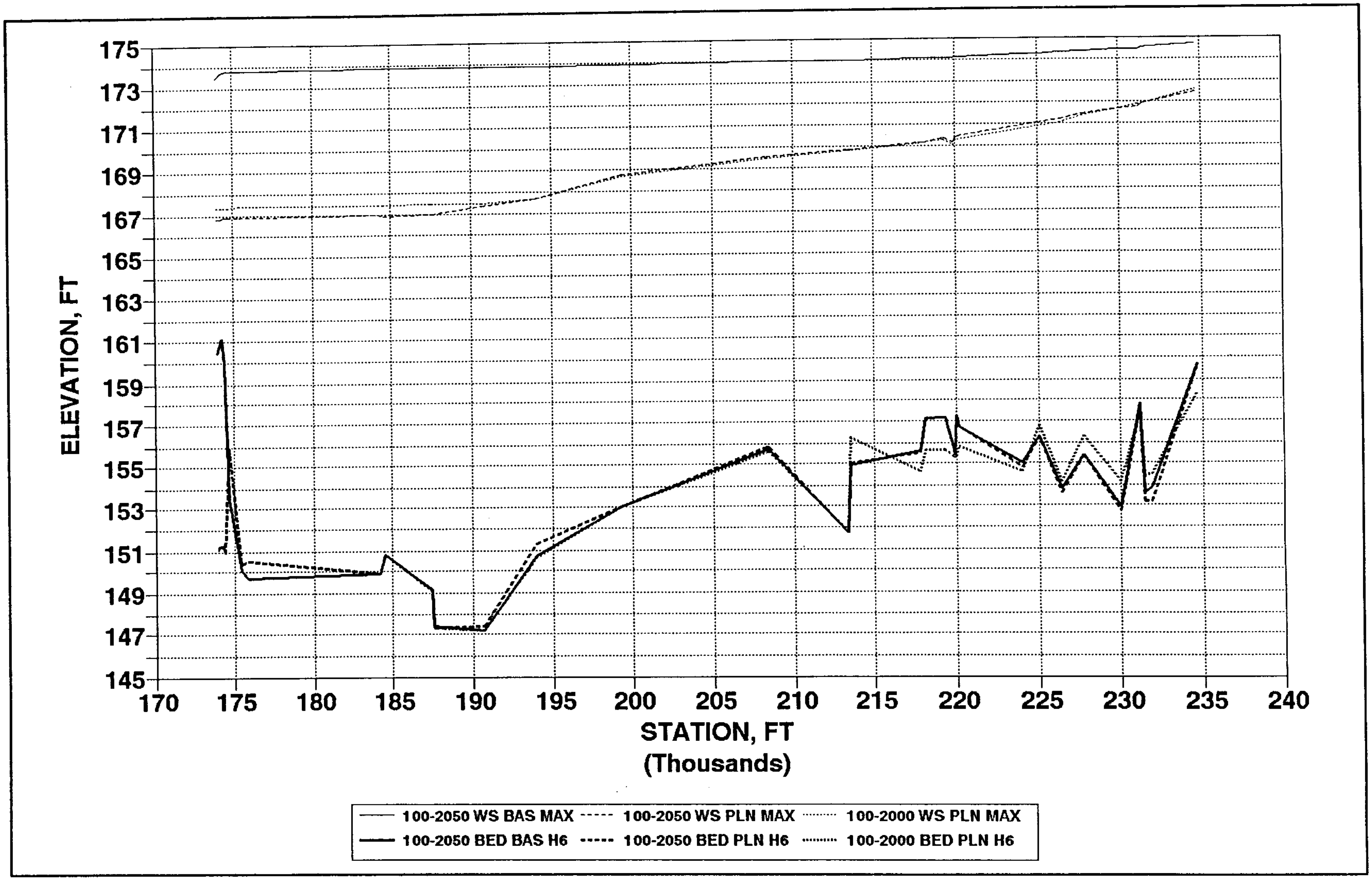

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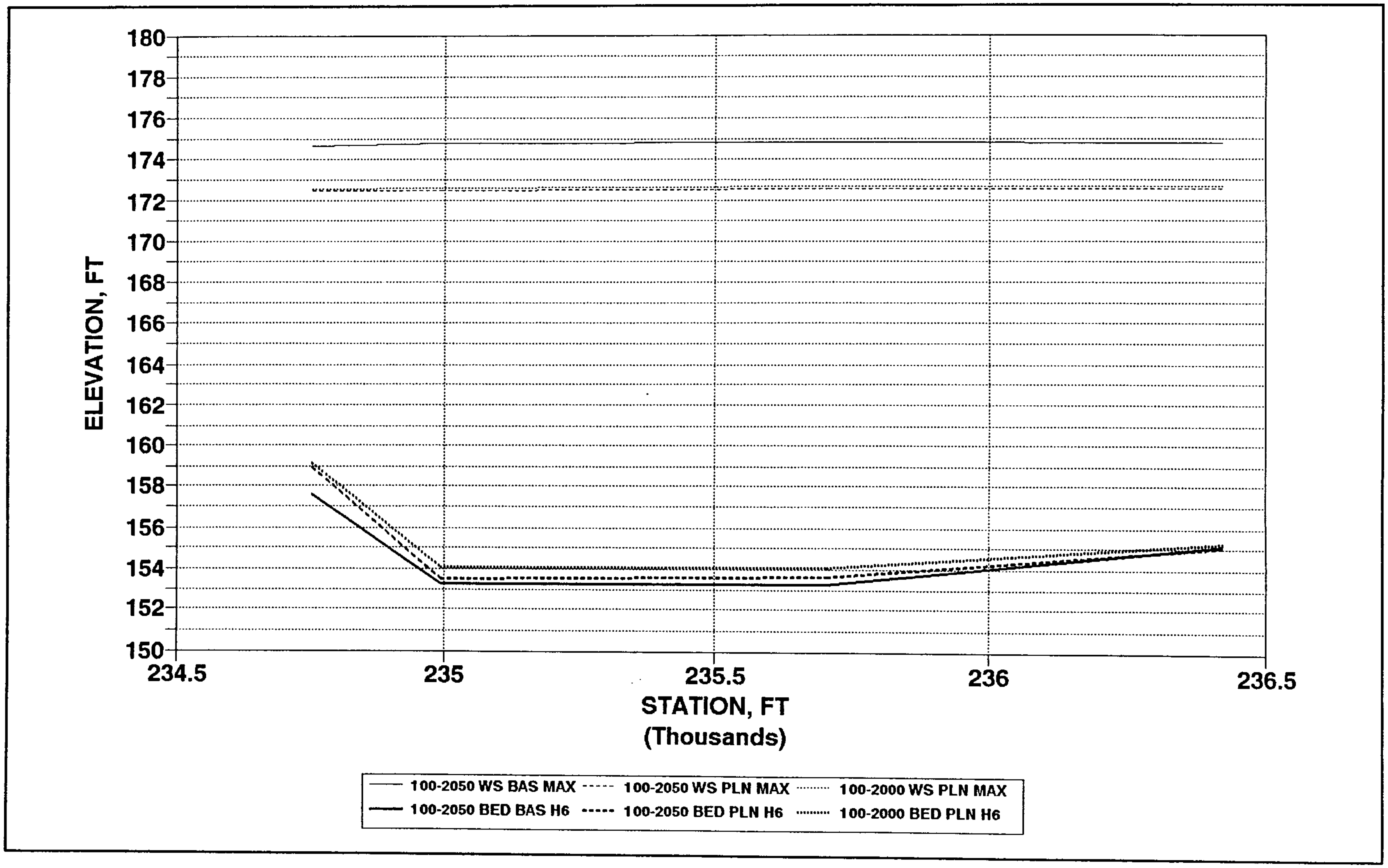

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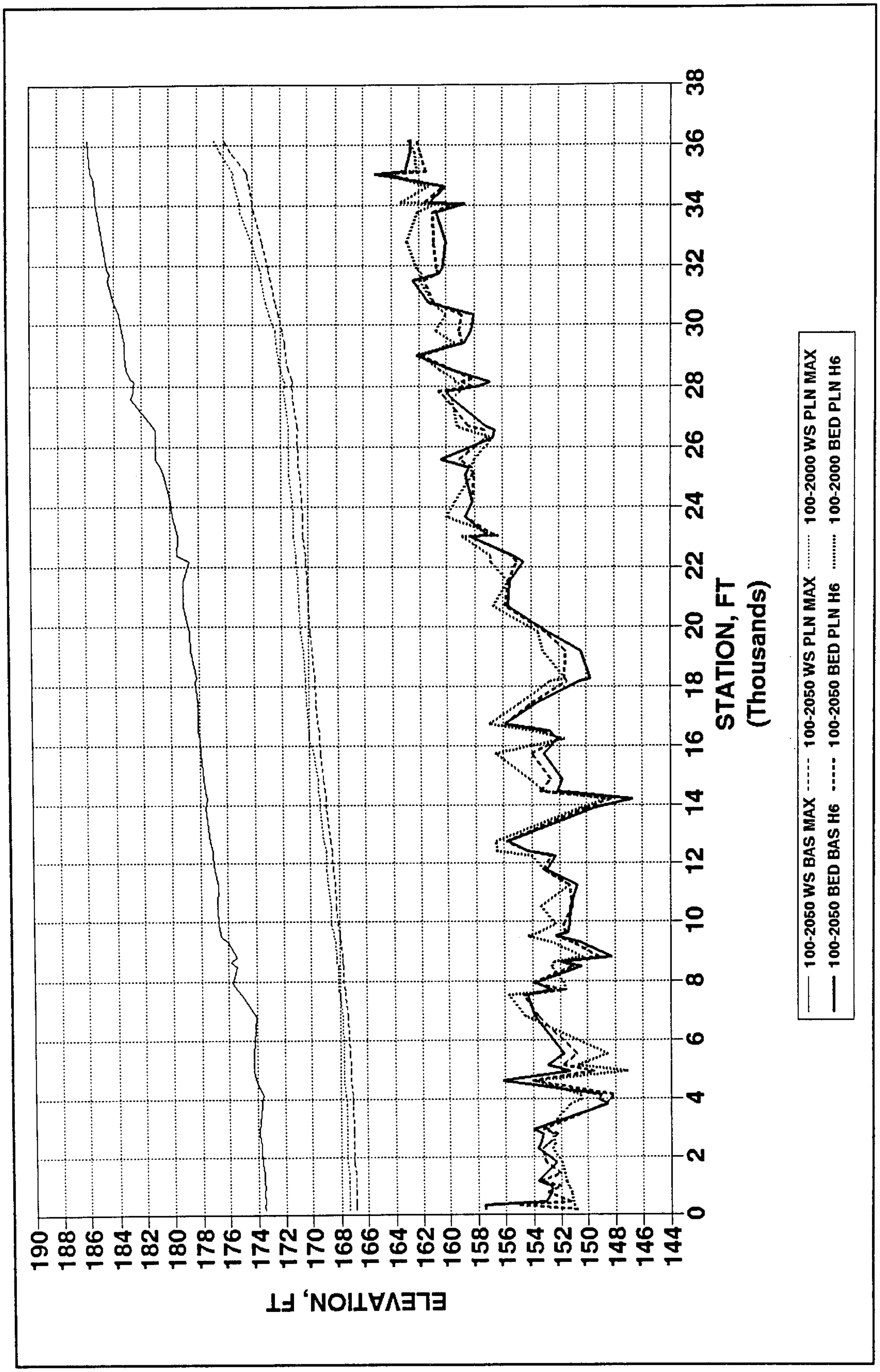

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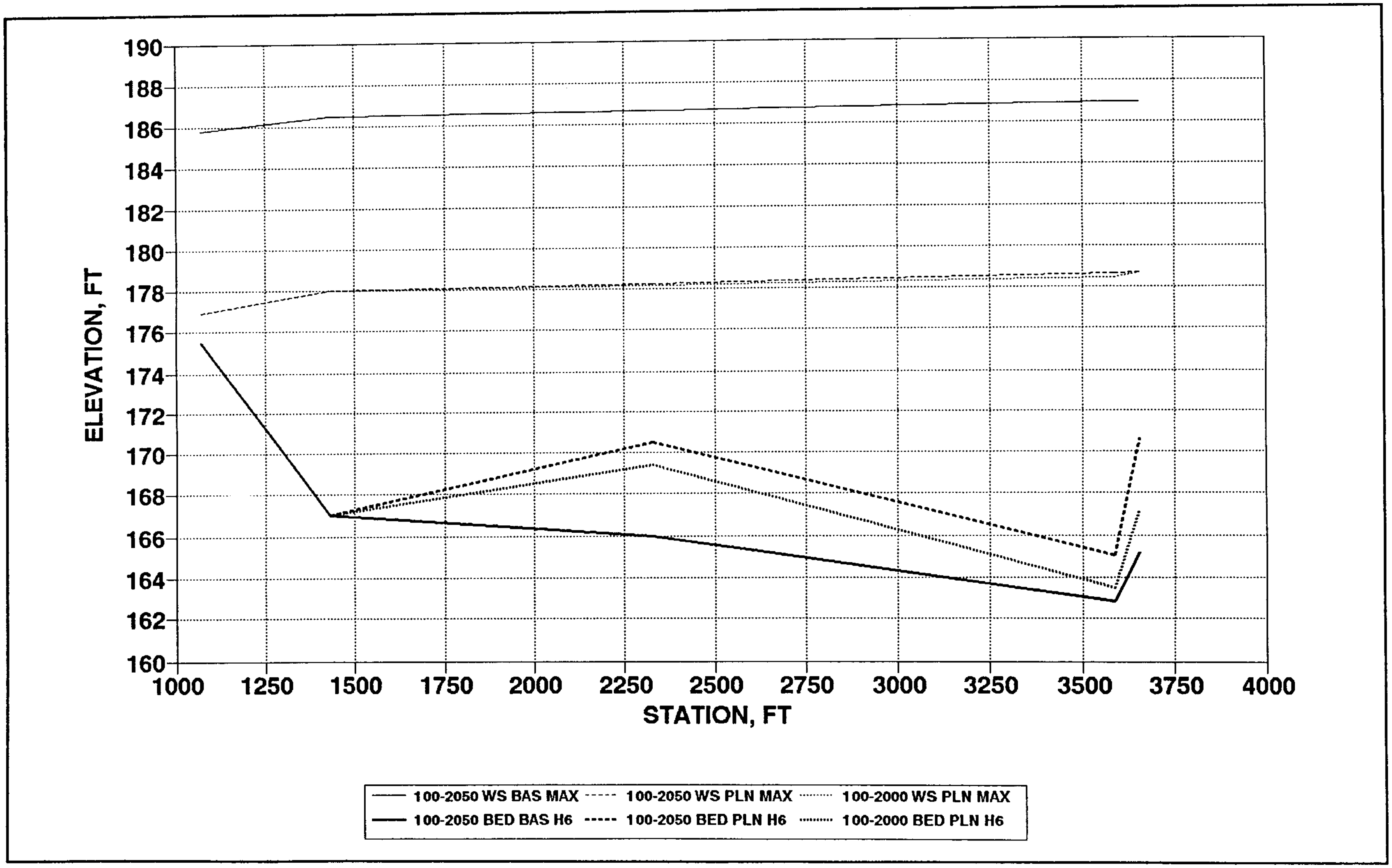

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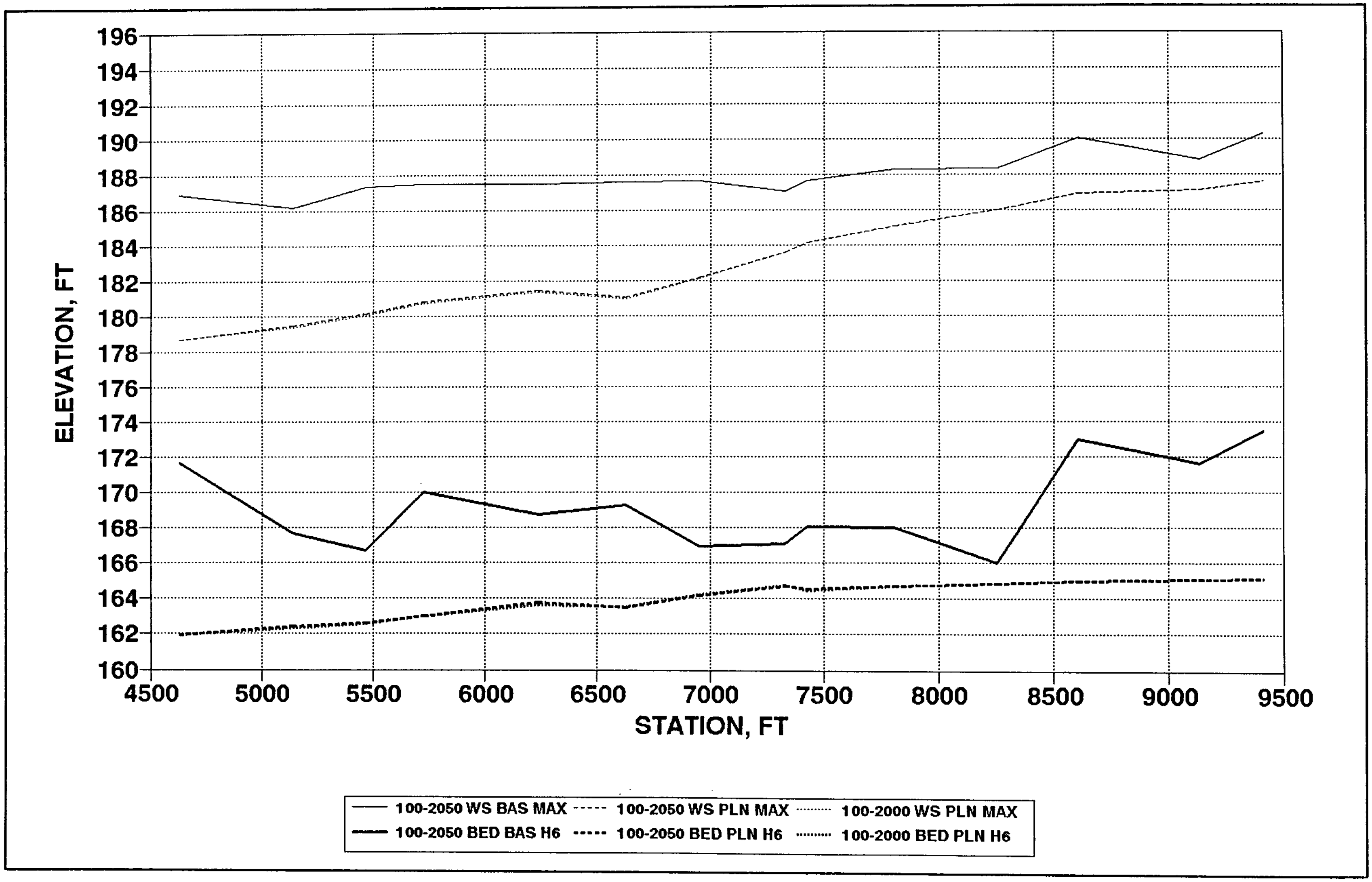

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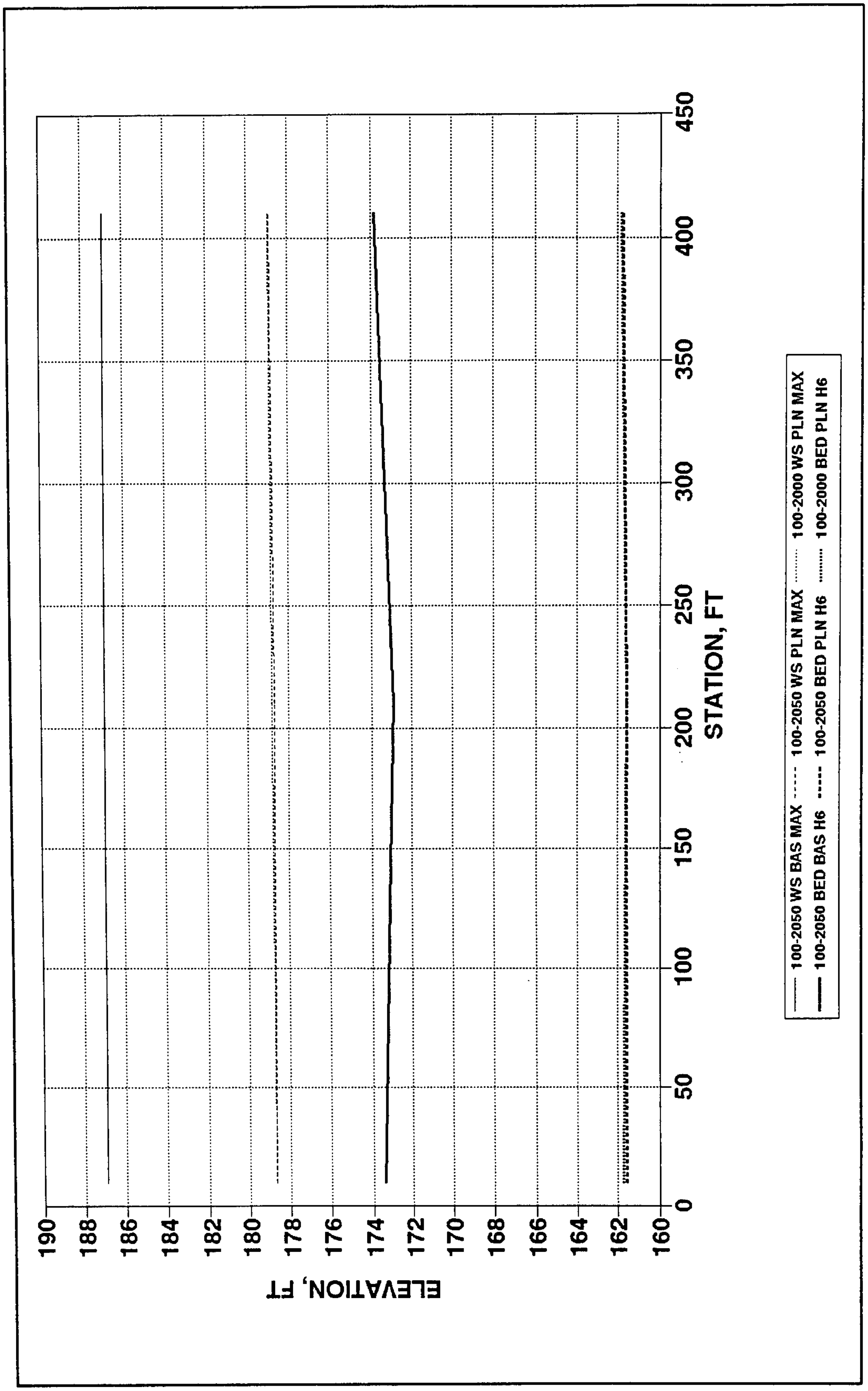

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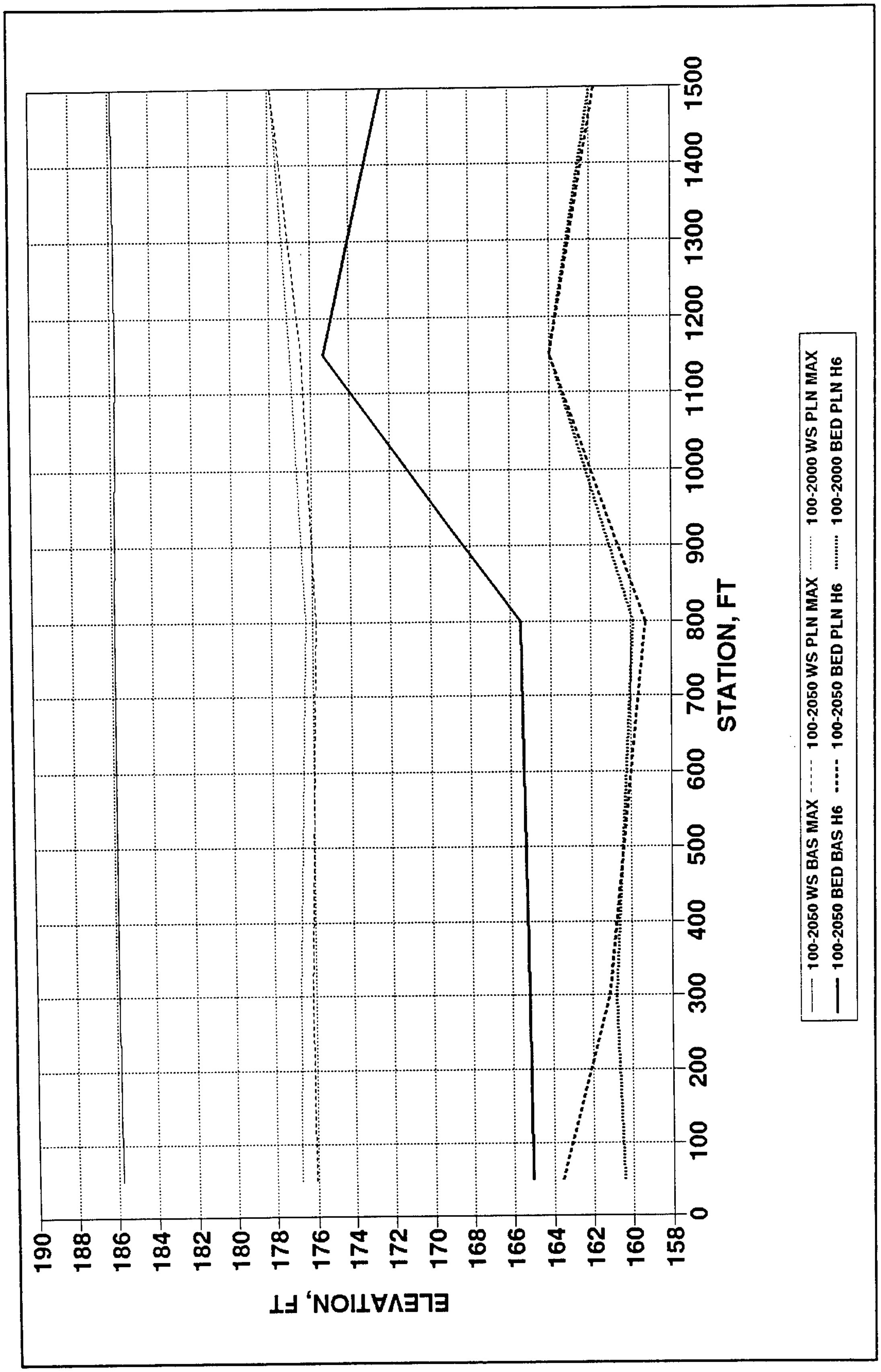

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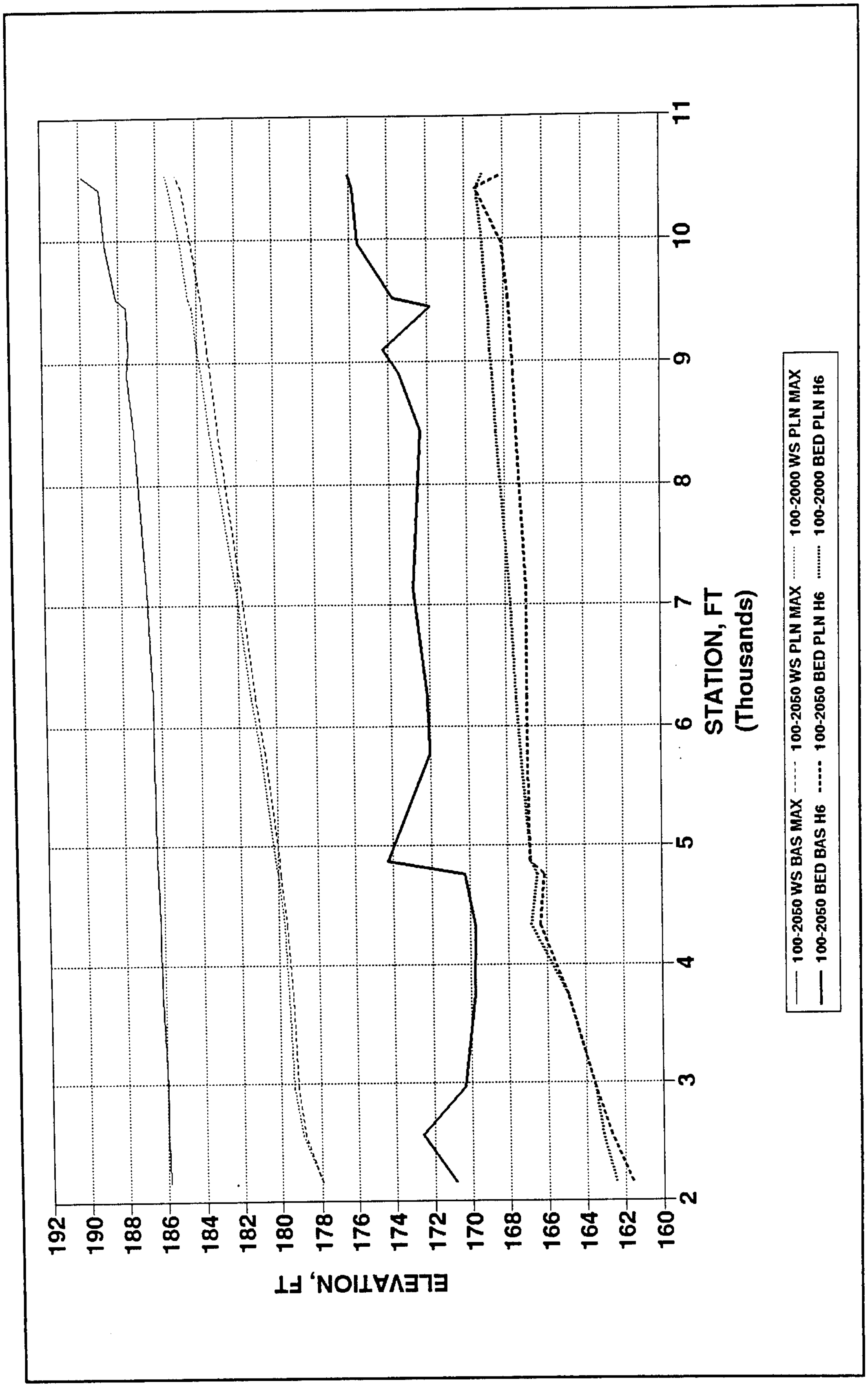

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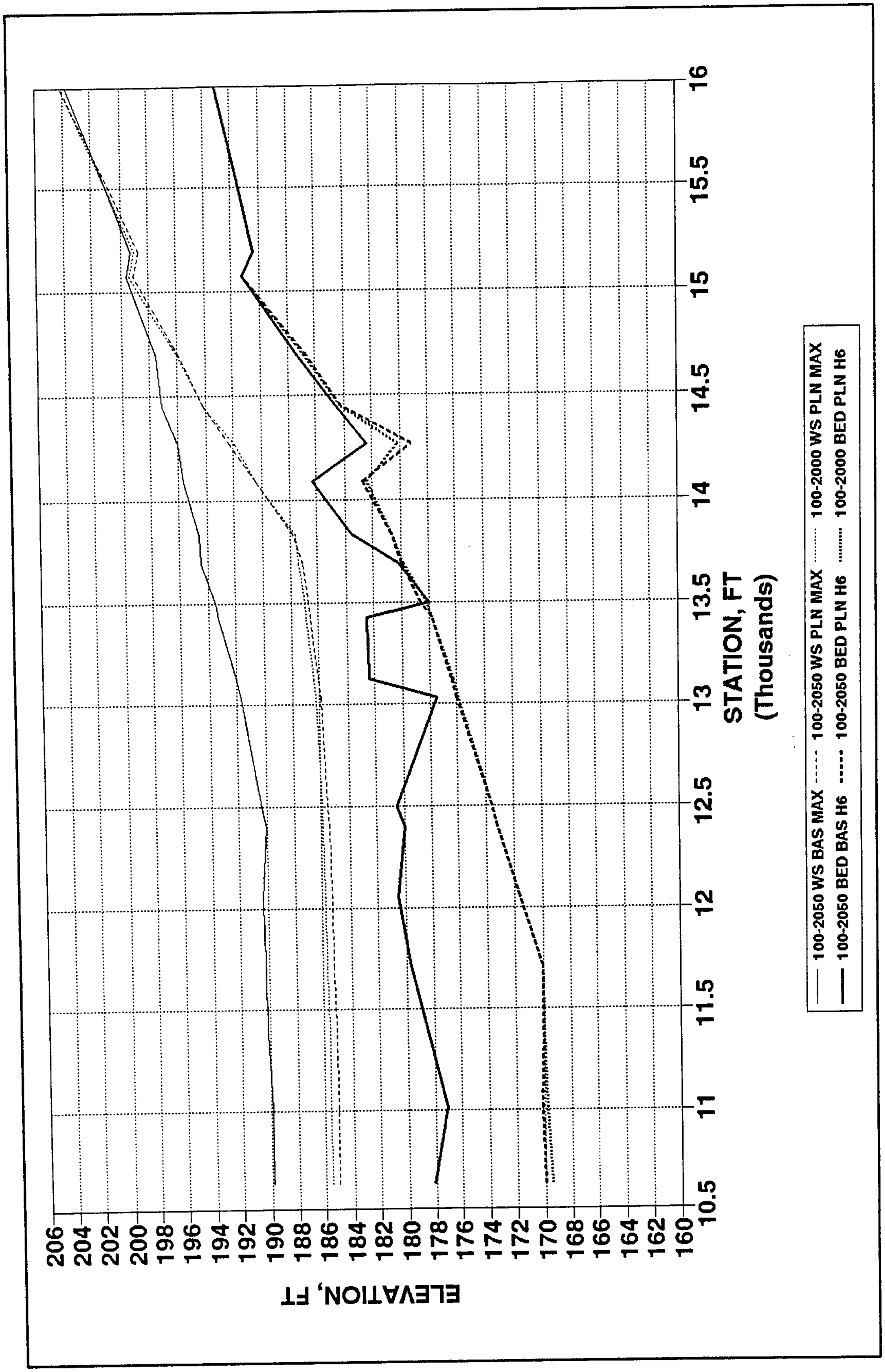

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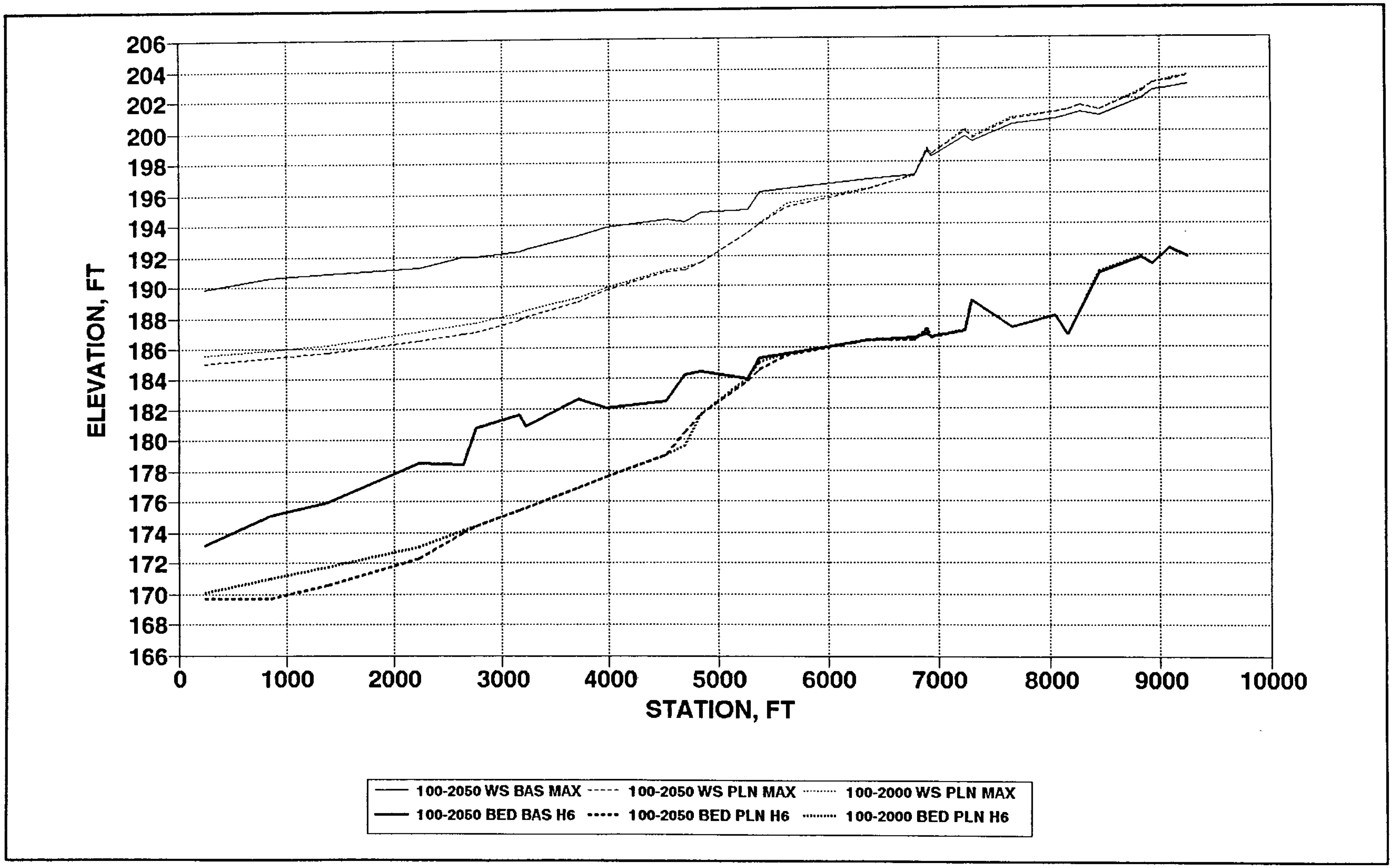

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13. ABSTRACT *(Maximum 200 words)*

A numerical model, HEC-6, was used to help determine the effects of the proposed Passaic River Tunnel Flood Protection Project in northern New Jersey. The modeling effort was one of the largest undertaken using HEC-6. A branching and closed loop network of 406 cross sections that included several locations in which reverse flows occur were some of the complicating factors that were dealt with in this study. The results of the study showed that the tunnel operation will not adversely affect sedimentation patterns in the network and should actually decrease the rate of present channel degradation in some reaches. Aggradation in the vicinity of Two Bridges and the Spur Tunnel Inlet could be a problem, which might require periodic maintenance.

14. SUBJECT TERMS

Aggradation	HEC-6	Sedimentation
Degradation	Model	Tunnel
Flood Protection	Numerical	

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